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INCLUDING PRESTRESSED CONCRETE



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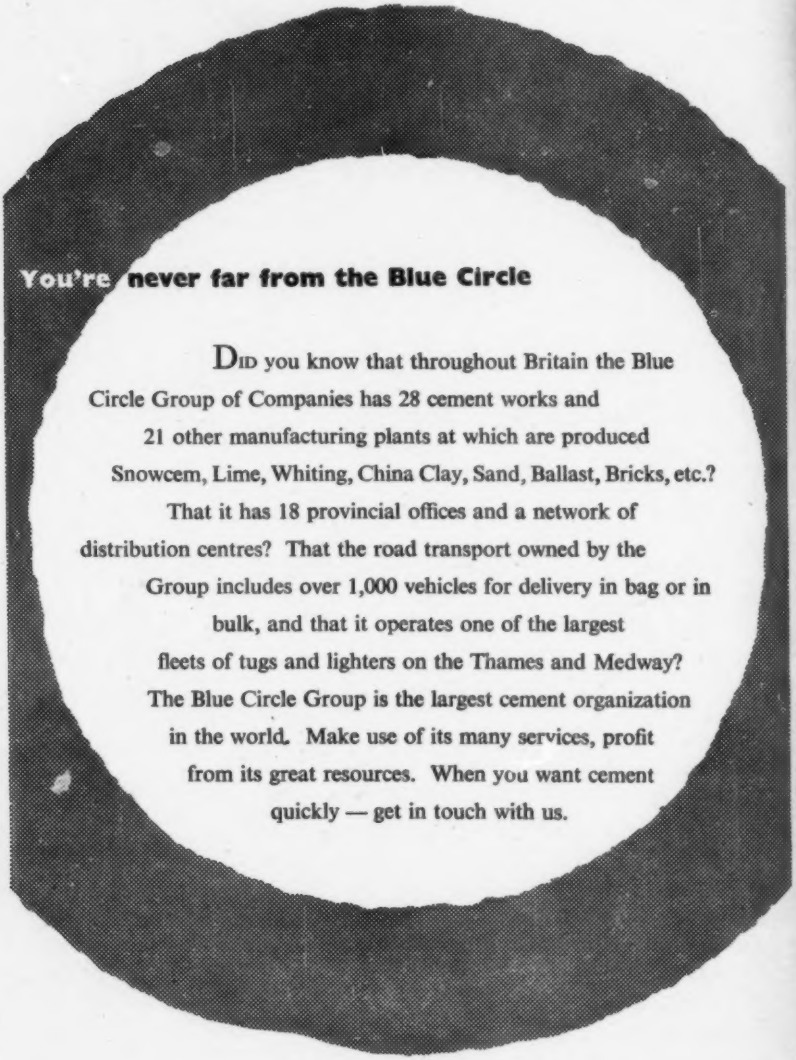
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CONCRETE
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INCLUDING PRESTRESSED CONCRETE

Volume LIII, No. 3. LONDON, MARCH, 1958.

# EDITORIAL NOTES

## Failure due to Combined Tension and Shear.

IN August, 1955, a large part of the roof of a United States Army warehouse collapsed. The roof was designed in accordance with the Building Code of the American Concrete Institute, the materials and workmanship were admitted to be good, the collapse occurred eighteen months after the building had been completed, and at the time the roof was supporting only its own weight. Nevertheless the collapse was found to be due to a weakness in the design. A review of other codes of practice, including the British Standard Code No. 114 (1948), showed that a design in accordance with the German code only would have averted the failure.

The collapse was due to the failure of an internal span of each of a series of long continuous beams at places where the calculations predicted a small bending moment that would cause sagging and a shearing stress of about 75 lb. per square inch. The nature of the failure, however, indicated that the beams had been subjected to large diagonal tensile stresses; it was found that the presence of direct tensile stresses, caused by shrinkage and changes of temperature and augmented by inefficient contraction joints, had so reduced the resistance of the beams to shearing forces that the structure failed. The American Concrete Institute revised its Building Code in 1956 with a view to preventing such failures.

It is known that conventional methods of designing reinforced concrete members subjected to shearing stresses are not always satisfactory. In a book published as long ago as 1924 * it was pointed out that whereas most reinforced concrete beams have a factor of safety of three against failure due to bending the factor of safety against failure due to shear is usually only two, that bent-up bars are better able to resist shearing forces than a system of closely-spaced stirrups, that conventional methods of design to resist shearing forces are safe only within limits, and that it is possible to design by such methods a beam that will be weak in its resistance to shear. The results of tests made in many countries in recent years have confirmed that stresses produced by shearing forces depend on many more factors than the shearing force, the breadth of the beam, and its lever arm, and it would have been desirable to have incorporated some recognition of this fact in the revised British Standard Code of Practice No. 114 of 1957.

* "Reinforced Concrete Design", by G. P. Manning.

The recommendations regarding shear in this Code differ from those of the Code issued in 1948 only in that the permissible stresses are increased ; the method of calculating shearing stresses is unchanged. In continuing to recommend this method it is still assumed that the relation between the tensile and compressive strengths of concrete is linear, that stirrups are as efficient as bent-up bars, and that the resistance of members to shearing forces is not affected by axial and bending stresses, the percentage of longitudinal reinforcement, the ratio of breadth to depth, the ratio of depth to span, or, in some cases, the complete absence of stirrups. All these assumptions are erroneous in some degree, the effects of axial and bending stresses being particularly important. The resistance of a member to shearing forces depends on the principal tensile stress in the concrete, which in turn depends on the shearing stresses, the stresses due to bending, and the axial stresses. In most cases the principal tensile stress is less than the shearing stress, since compressive stresses due to bending usually reduce the principal tensile stress and axial tensile stresses are generally negligible ; in these circumstances the shearing stress is a safe measure of the effect of shearing forces. At the points of contraflexure of long continuous beams, however, where stresses due to bending are small and the axial tensile stresses caused by shrinkage may be appreciable, the principal tensile stress may often exceed the shearing stress, and the United States experience shows that an extreme, though not impossible, combination of adverse conditions can lead to collapse.

Most structures designed in accordance with the recommendations of the Code are safe. It is true that the compilers of the Code assume that the design of reinforced concrete will be entrusted to competent engineers, but even if this assumption were correct a document in which such elementary statements are made as " plane sections may be assumed to remain plane " could be expected to include a warning of circumstances in which its recommendations may result in an unsafe structure. There is an increasing tendency for engineers to substitute for their own skill and judgment a reliance on recommendations embodied in codes of practice ; this is to be deplored, but its existence should be recognised and the compilers of codes should reduce as much as possible any chance of their recommendations being misapplied. It would not be difficult to incorporate recommendations that would remove the risk of such failures. For example, the Code of the American Concrete Institute now limits the permissible shearing stress in beams with no web reinforcement to 90 lb. per square inch or 3 per cent. of the specified compressive strength at 28 days, whichever is the lesser, and requires that sufficient web reinforcement to resist two-thirds of the shearing force be provided in continuous or restrained beams, extending from the support to a distance equal to either one-sixteenth of the clear span or the depth of the beam (whichever is the lesser) past the extreme position of the point of contraflexure. This reinforcement must be provided even if the shearing stress is less than that permissible ; the requirement does not apply to flanged beams.

The risk that may accompany a thoughtless application of the recommendations of the Code with regard to shear may become of greater importance in view of the increasing use of concrete with high strength and high cement content and it is regrettable that this problem is not dealt with in such an important document.

# The London–Yorkshire Motorway.

One of the major new roads proposed by the Government in the year 1955 was one 70 miles long from St. Albans to Rugby, at an estimated cost of £211,000 per mile. A contract has now been placed for the construction of 53 miles of this road extending from the end of the St. Albans by-pass road to Dunchurch by-pass road near Rugby. Tenders were invited for four separate contracts of between ten and sixteen miles each, with separate prices for the completion of the work by October 1959 and by October 1960, and for alternative forms of construction of the carriageway. The whole of the contract has been awarded to Messrs. John Laing & Son, Ltd., for completion by October 31, 1959, at a cost of £15,000,000, or about £283,000 a mile.

The new road will be carried over or under all existing roads. At a few places, where important highways are crossed, access roads will be provided to enable traffic from existing roads to enter the motorway without crossing it. There will be six two-level junctions, forty-six bridges over public roads, eight bridges over railways, three bridges over canals, and five over rivers, and many smaller bridges for public paths and for private and agricultural use. At intervals of about twelve miles there will be service areas on both sides of the motorway. These areas, of roughly circular shape, will provide space for petrol stations, parking areas, picnic sites, restaurants, and other purposes, and they will be linked by footbridges.

It will be necessary to excavate some 10,000,000 cu. yd. of earth and to lay about 2,400,000 sq. yd. of roadway. Some 300,000 cu. yd. of concrete and about 12,000 tons of reinforcement will be required for the bridges.

The road will be for motor vehicles only. There will be no waiting on the road, but drivers will be able to stop in an emergency on verges formed alongside

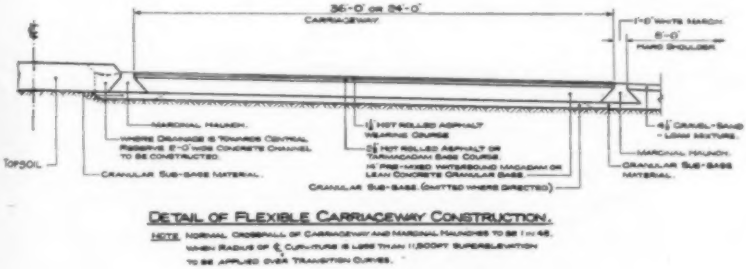



**Fig. 1.—Cross Section of Road.**

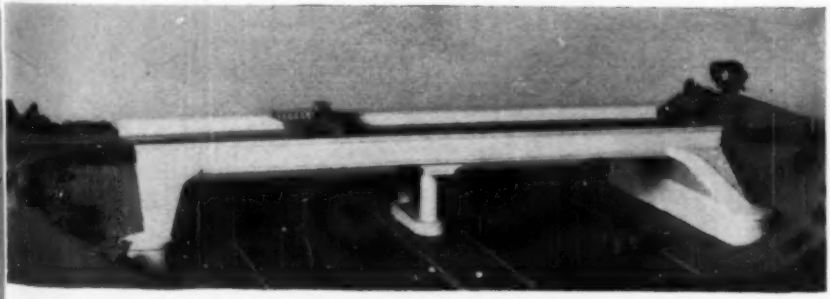


**Fig. 2.—Design for an Over-bridge.**

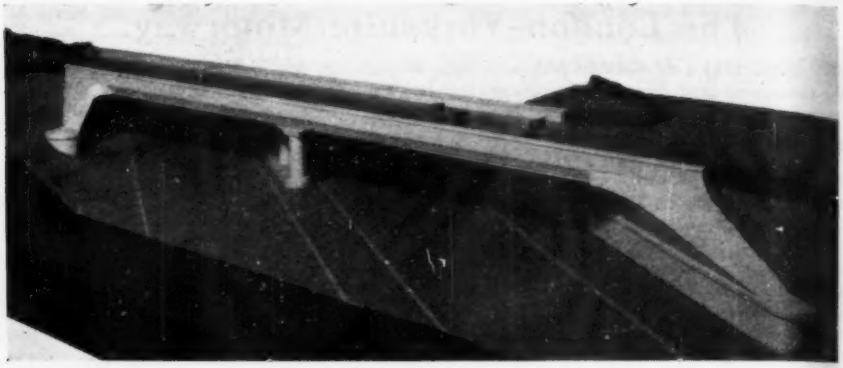


Fig. 3.—A Skew Over-bridge.

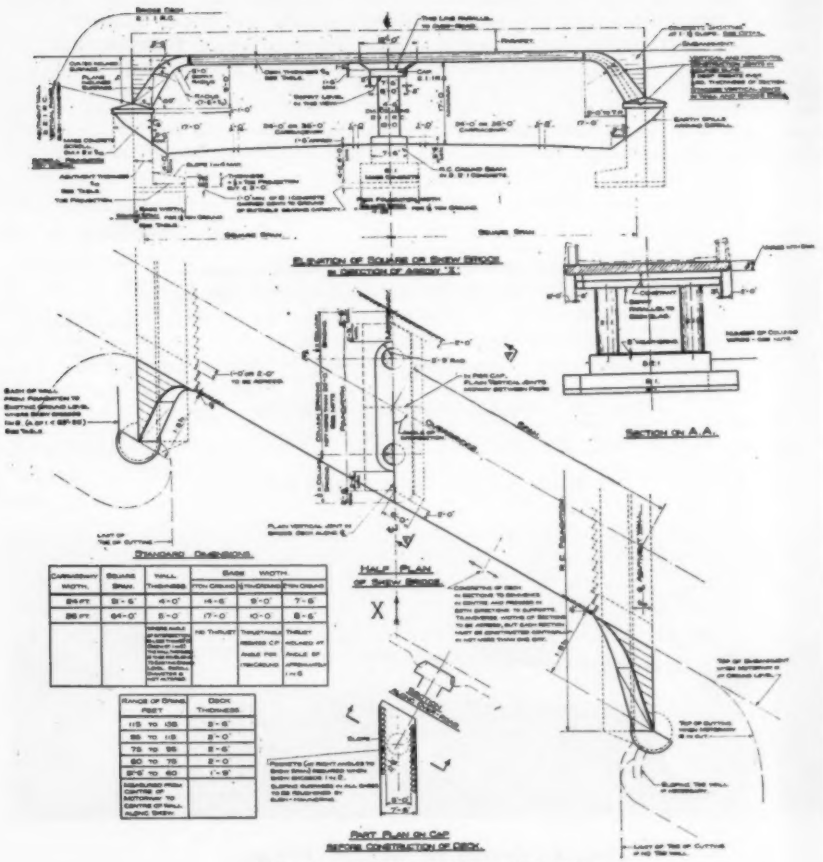

ELEVATION OF SQUARE OR SKEW BRIDGE IN DIRECTION OF ARROW X
Section on A.A.
Half Plan of Skew Bridge
Standard Dimensions.
Part Plan on Cap showing construction of deck.

Fig. 4.—Details of Over-bridges.

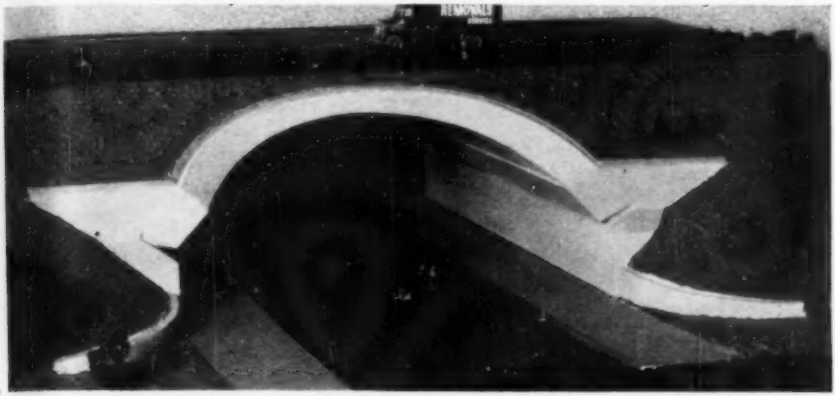


Fig. 5.—An Arched Under-bridge.

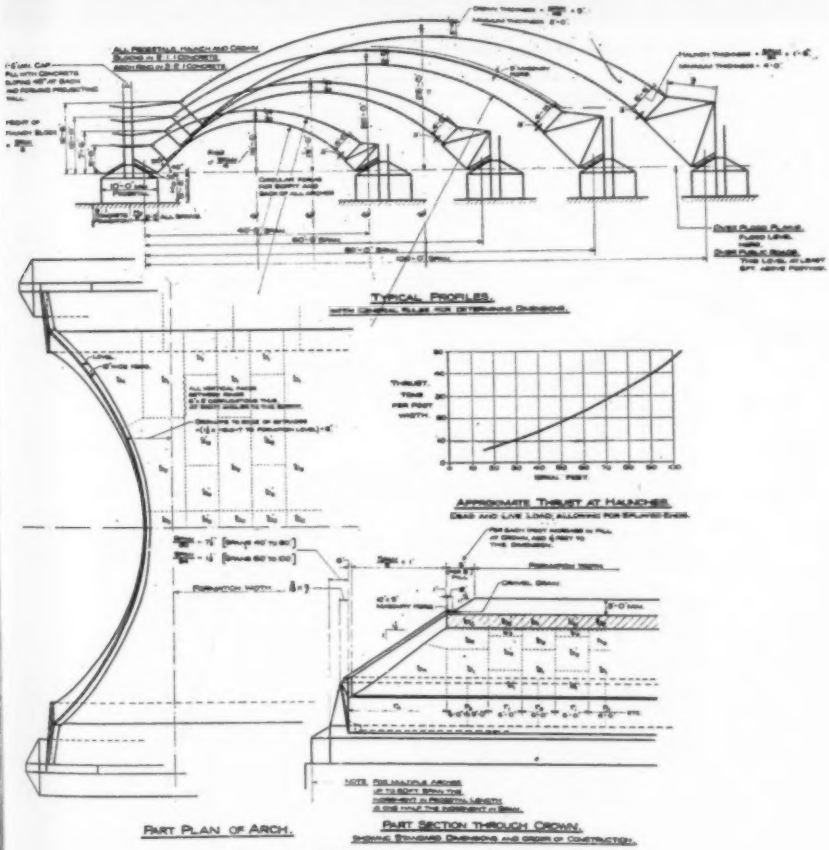



Fig. 6.—Details of Arched Under-bridges.

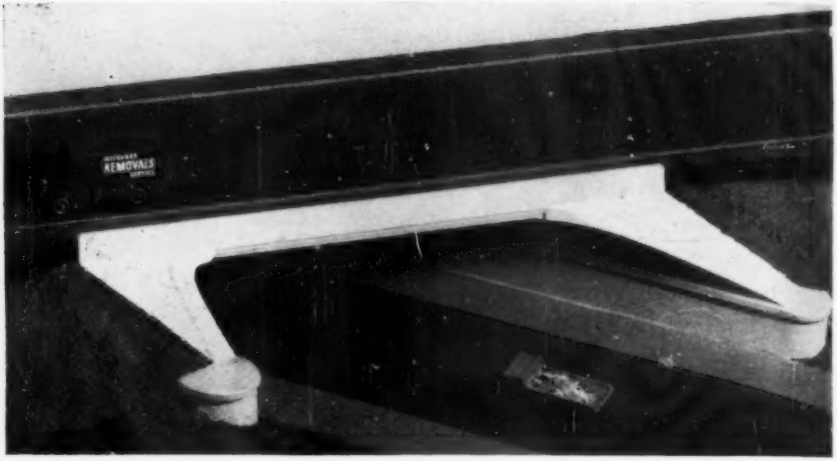
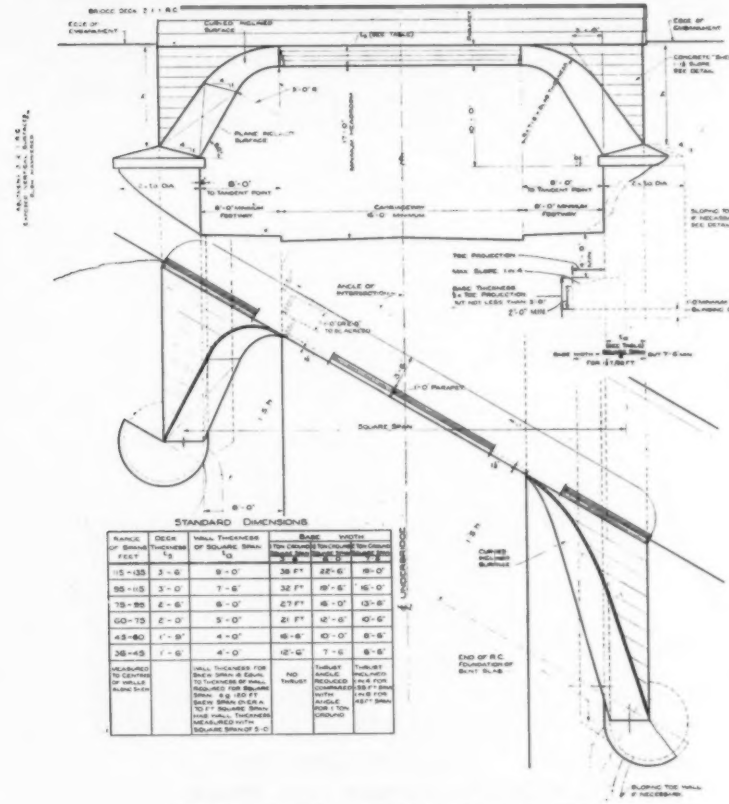
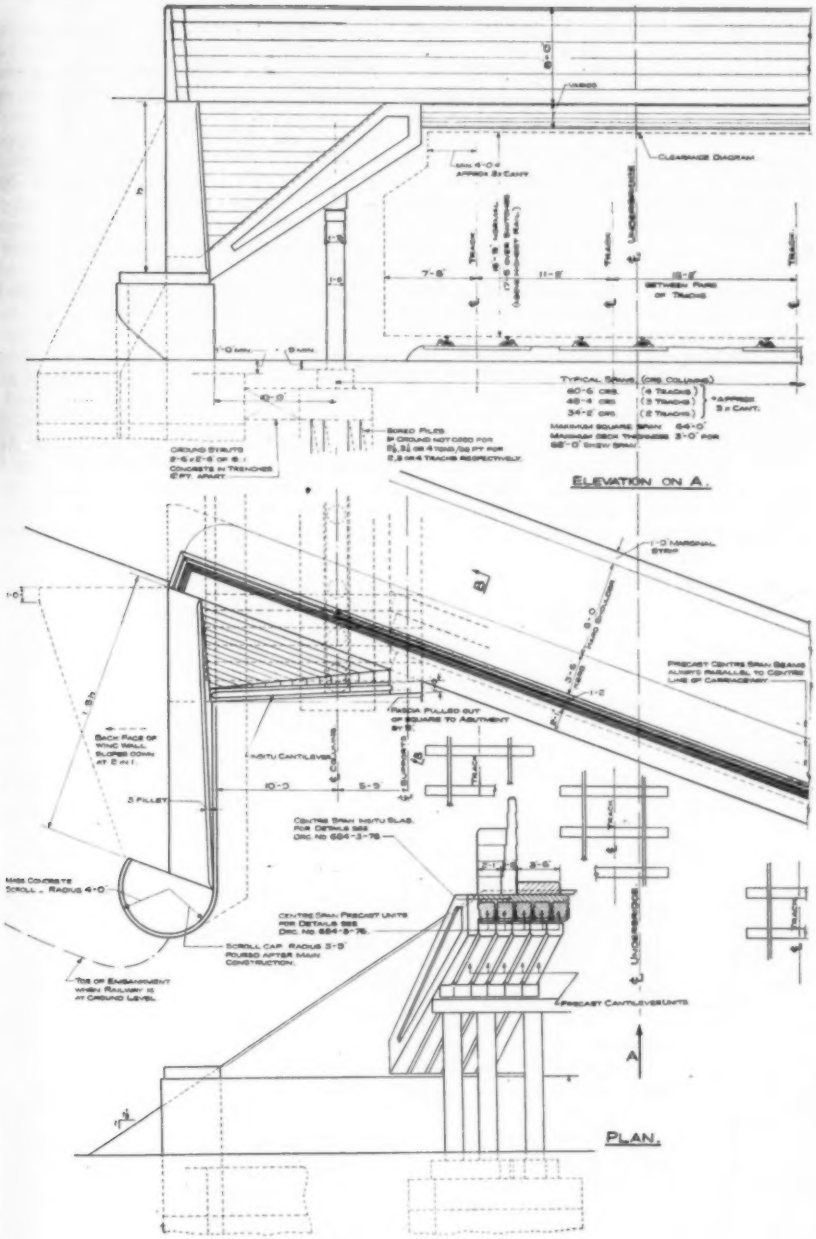



Fig. 7.—Under-bridge for Use where Headroom is Restricted.




| RANGE OF SPANS FEET | DECK THICKNESS $t_D$ | WALL THICKNESS OF SQUARE SPAN $t_D$ | BASE WIDTH 10 TONS/SQ FT SQUARE SPAN | 12 TONS/SQ FT SQUARE SPAN | 15 TONS/SQ FT SQUARE SPAN |
|---|---|---|---|---|---|
| | | | $\frac{1}{3}$ H | $\frac{1}{4}$ H | $\frac{1}{5}$ H |
| 115 - 135 | 3' - 6" | 9' - 0" | 38 FT | 28' - 6" | 19' - 0" |
| 85 - 115 | 3' - 0" | 7' - 6" | 32 FT | 24' - 0" | 16' - 0" |
| 75 - 85 | 2' - 6" | 6' - 0" | 25 FT | 18' - 9" | 12' - 6" |
| 60 - 75 | 2' - 0" | 5' - 0" | 21 FT | 15' - 6" | 10' - 6" |
| 45 - 60 | 1' - 9" | 4' - 0" | 16' - 6" | 12' - 0" | 8' - 0" |
| 30 - 45 | 1' - 6" | 4' - 0" | 12' - 6" | 9' - 6" | 6' - 6" |
| MEASURED TO CENTRE OF WALLS ALONG SKEW | | WALL THICKNESS FOR SKEW SPAN IS EQUAL TO THICKNESS OF WALL REQUIRED FOR SQUARE SPAN E.G. 100 FT SKEW SPAN OVER A 70 FT SQUARE SPAN HAS WALL THICKNESS MEASURED WITH SQUARE SPAN OF 5'-0 | NO THRUST | THRUST ANGLE REDUCED COMPARED WITH ANGLE FOR 1 TON GROUND | THRUST INCLINED 1/4 H FOR 35 FT SPAN 1/5 H FOR 45 FT SPAN |

Fig. 8.—Details of Bridge shown in Fig. 7.


CLEARANCE DIAGRAM
TRACK
UNDERBRIDGE
BETWEEN PAIRS OF TRACKS
TYPICAL SPANS (PRE COLLING)
60'-0" CRS. (4 TRACKS)
48'-4" CRS. (3 TRACKS)
34'-6" CRS. (2 TRACKS)
*APPROX. 3½ CANT.
MAXIMUM SQUARE SPAN 64'-0"
MAXIMUM DECK THICKNESS 3'-0" FOR 68'-0" SKEW SPAN
BORED PILES
GROUND STRUTS
CONCRETE IN TRENCHES
ELEVATION ON A.
1'-0 MARGINAL STRIP
PRECAST CENTRE SPAN BEAMS ALIGNED PARALLEL TO CENTRE LINE OF CARRIAGEWAY
BACK FACE OF WING WALL SLOPES DOWN AT 2 in 1
IN-SITU CANTILEVER
3 FILLET
CENTRE SPAN IN-SITU SLAB.
CENTRE SPAN PRECAST UNITS
SCROLL CAP RADIUS 3'-9"
TOE OF EMBANKMENT WHERE RAILWAY IS AT GROUND LEVEL
PRECAST CANTILEVER UNITS
PLAN.

SECTION B.B.

**Fig. 9.—Bridge with Precast Beams.**

each of the two carriageways. Each carriageway will be 36 ft. wide, except for short lengths where the width will be 24 ft. At the peak of construction about 3000 men will be employed on the work.

The Ministry of Transport and Civil Aviation is the authority responsible for the work. Sir Owen Williams & Partners, by whose courtesy the following details are given, are the consulting engineers.

The carriageway will be of "flexible" construction (*Fig.* 1), comprising a base of granular material followed by 14 in. of pre-mixed waterbound macadam or lean concrete, 2½ in. of tarmacadam, and a surface of hot-rolled asphalt 1½ in. thick.

## Bridges.

More than a hundred bridges will be built, and where possible standard designs have been adopted. *Figs.* 2 and 3 show models of standard square and skew over-bridges, and the arrangement and dimensions are shown in *Fig.* 4. In order to keep the construction depth as small as possible, and thereby reduce the cost of the embankments, the decks are designed as slabs. The central support consists of reinforced concrete columns, 4 ft. 6 in. in diameter, supported by a plain concrete foundation and capped with a splayed reinforced concrete beam. The abutments are curved in plan and in elevation to form retaining walls.

The adoption of a standard design for all the under-bridges would have been uneconomical because of the wide variations in the spans and heights. Where the height is sufficient, three-hinged arch bridges of plain concrete (*Figs.* 5 and 6) will be built. The cross section of the arch varies in such a way that the position of the line of thrust is the same when the arch is supporting only its own weight and when it is also supporting the filling material. The arch will be constructed

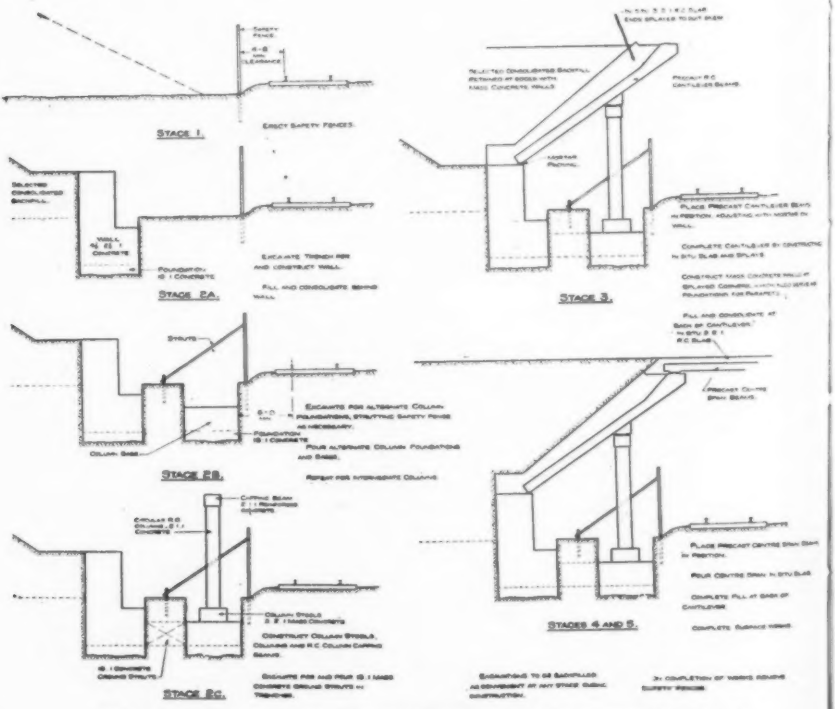



**Fig. 10.—Sequence of Construction of Bridge shown in Fig. 9.**

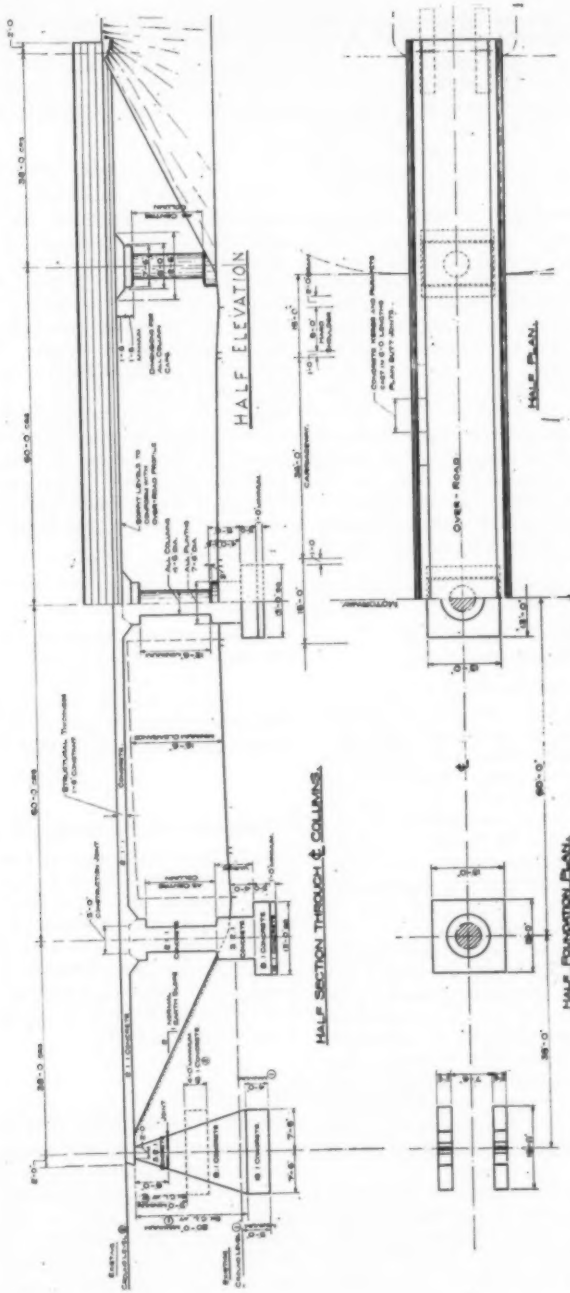



Fig. 11.—Details of Minor Over-bridges.

in segments, the sequence of construction being arranged to reduce as much as possible the effects of shrinkage.

Where the height is insufficient to allow the use of arches, the type of bridge shown in *Figs.* 7 and 8 will be used. The design of this is similar to that of the over-bridges, with the omission of the central supports.

Where the road passes over railways, bridges of the type shown in *Fig.* 9 will be used; precast reinforced concrete will be used in these cases to keep the height over the motorway as small as possible. The central span will comprise precast beams parallel with the centre-line of the road and supported on the ends of precast cantilevers placed normal to the centre-line of the railway. The cantilevers will be supported by reinforced columns and plain concrete retaining walls. Slabs will be cast in place on top of the precast members, and the stability of the cantilevers will be ensured by the weight of the earth placed on them. The sequence of construction is shown in *Fig.* 10.

Most of the minor over-bridges will also be of standard types; an over-bridge suitable for light traffic is shown in *Fig.* 11.

### St. Albans By-pass Road.

A contract has also been placed for the first part, twelve miles long, of the St. Albans by-pass road, which will extend the new St. Albans–Rugby road nearer to London. There will be two carriageways, partly 36 ft. wide and partly 24 ft. wide, of reinforced concrete. The work has been designed by the Hertfordshire County Council for the Ministry of Transport and Civil Aviation; there will be thirty-nine bridges of which twenty-six have been designed by the County Council and thirteen by Sir Owen Williams & Partners. The contractors are Tarmac Civil Engineering, Ltd. The work is to be completed by October 31, 1959.

---

### The Fire Resistance of Prestressed Concrete.

TESTS to determine the effect of temperatures of 300 deg. C. to 600 deg. C. on the strength of steel in reinforced and prestressed concrete members are described by Messrs. Dannenberg, Deutschmann, and Melchior in Bulletin No. 122 of Deutscher Ausschuss für Stahlbeton (Berlin: Wilhelm Ernst & Sohn. Price 14 D.M.). Their conclusions are as follows.

(1) The fire resistance of prestressed concrete members is less than that of reinforced concrete members.

(2) Even relatively moderate rises of temperature, ranging from 200 deg. C. to 300 deg. C., result in appreciable losses of prestress due to the thermal expansion of the steel. This leads to considerable deflections which do not entirely disappear after the steel has cooled. The prestressed member loses its prestress and becomes an ordinary reinforced concrete member with highly-stressed untensioned reinforcement.

(3) Progressive heating of the tensioned steel results in further expansion accompanied by a gradual reduction of the depth of the compressive zone until the concrete is no longer able to sustain the compressive force and the collapse of the member becomes imminent.

(4) To avoid such damage, prestressing steel should have a substantial cover of concrete, or be provided with an additional layer of fire-resisting material.

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### Congress on Prestressed Concrete.

THE Cement and Concrete Association has made arrangements for delegates from Great Britain to attend the third Congress of the International Federation of Prestressing, to be held in Berlin from May 5 to May 10. Details may be had from the Secretary, Prestressed Concrete Development Group, 52 Grosvenor Gardens, London, S.W.1.

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### Grants for Students and Research.

A BROCHURE describing the grants made available for students by the Department of Scientific and Industrial Research has been published (price 1*s.* 6*d.*) by H.M. Stationery Office with the title "Notes on D.S.I.R. Grants for Graduate Students and Research Workers".

# Design of Eccentrically-Loaded Columns by the Load-Factor Method.

## 3.—Unsymmetrical Cold-worked Reinforcement.*

By J. D. BENNETT, B.Eng.

THE basis of design and the notation used in this article are fully stated in the first article in this series (see this journal for November, 1957) and conform to the recommendations of British Standard Code of Practice No. 114 (1957). In this article equations and graphs are derived for columns with unsymmetrical cold-worked reinforcement, and the charts include the whole range of combined bending and compression. As in the case of previous charts, no allowance is made for the reduction in the area of concrete due to the compressive steel, but this error is small; for a column with 6 per cent. of steel and 1 : 2 : 4 concrete the greatest error is 2·2 per cent. when the column is concentrically loaded. For columns with eccentric loads and smaller percentages of steel the error is less. In addition to the notation given previously, the following are used:

$A_s$, total cross-sectional area of steel $= rbd$.
$A_{sc}$, cross-sectional area of steel in compression $= r_2bd = xrbd$.
$A_{st}$, cross-sectional area of steel in tension $= r_1bd = (1 - x)rbd$.
$2p_t$, stress in tensile steel at ultimate load. It is important to note that $2p_t$ is used in this article where $p_t$ was used in the first article; $p_t$ is a variable stress with a maximum value of $p_{st}$.

$$r_1 + r_2 = r.$$

At the balanced condition, between tensile and compressive failure, the maximum strain in the concrete occurs simultaneously with the maximum stress in the tensile steel.

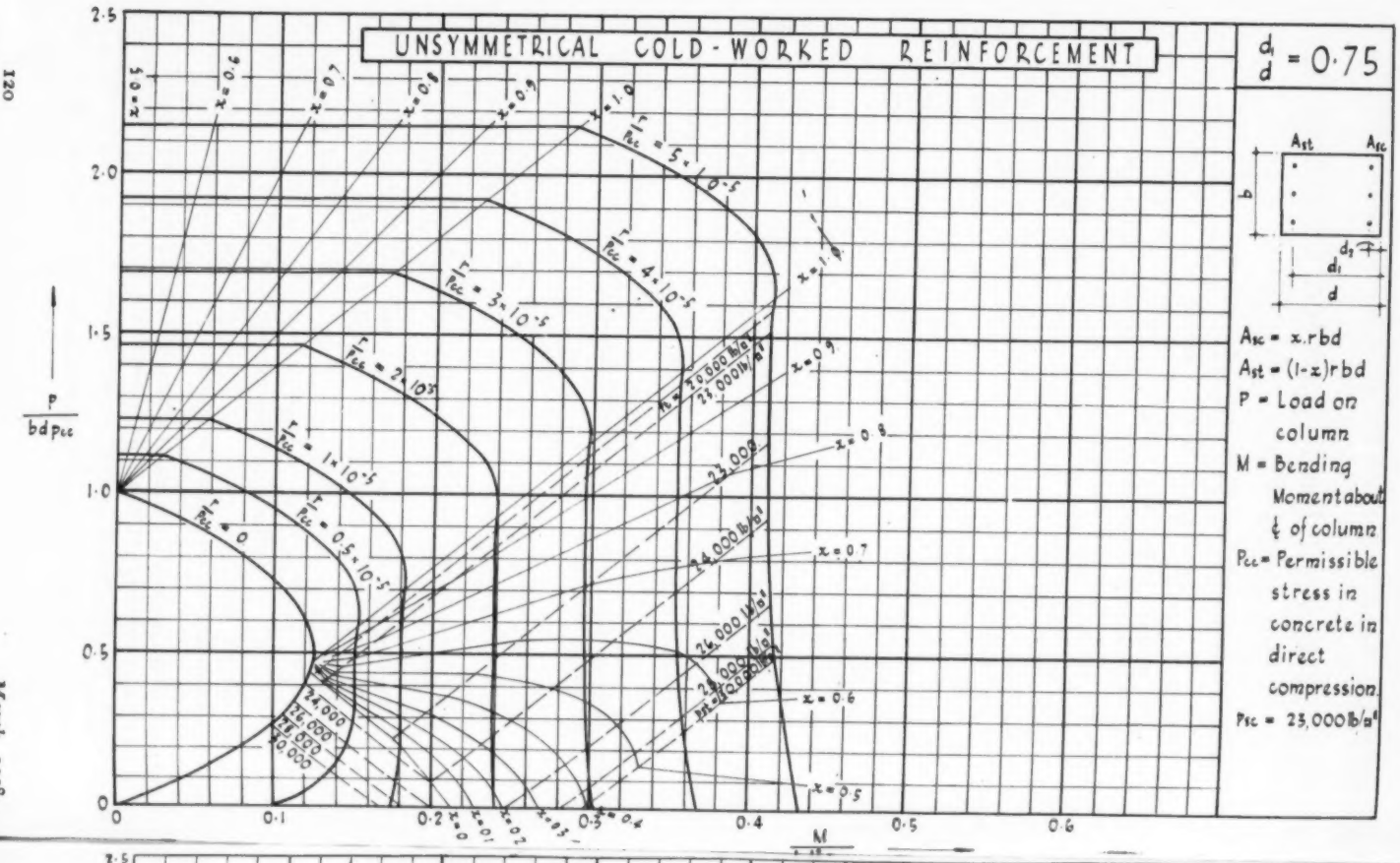
From the linear strain relationship, $\dfrac{2p_{st}}{E_s(1 - n)} = \dfrac{1}{300n}$.

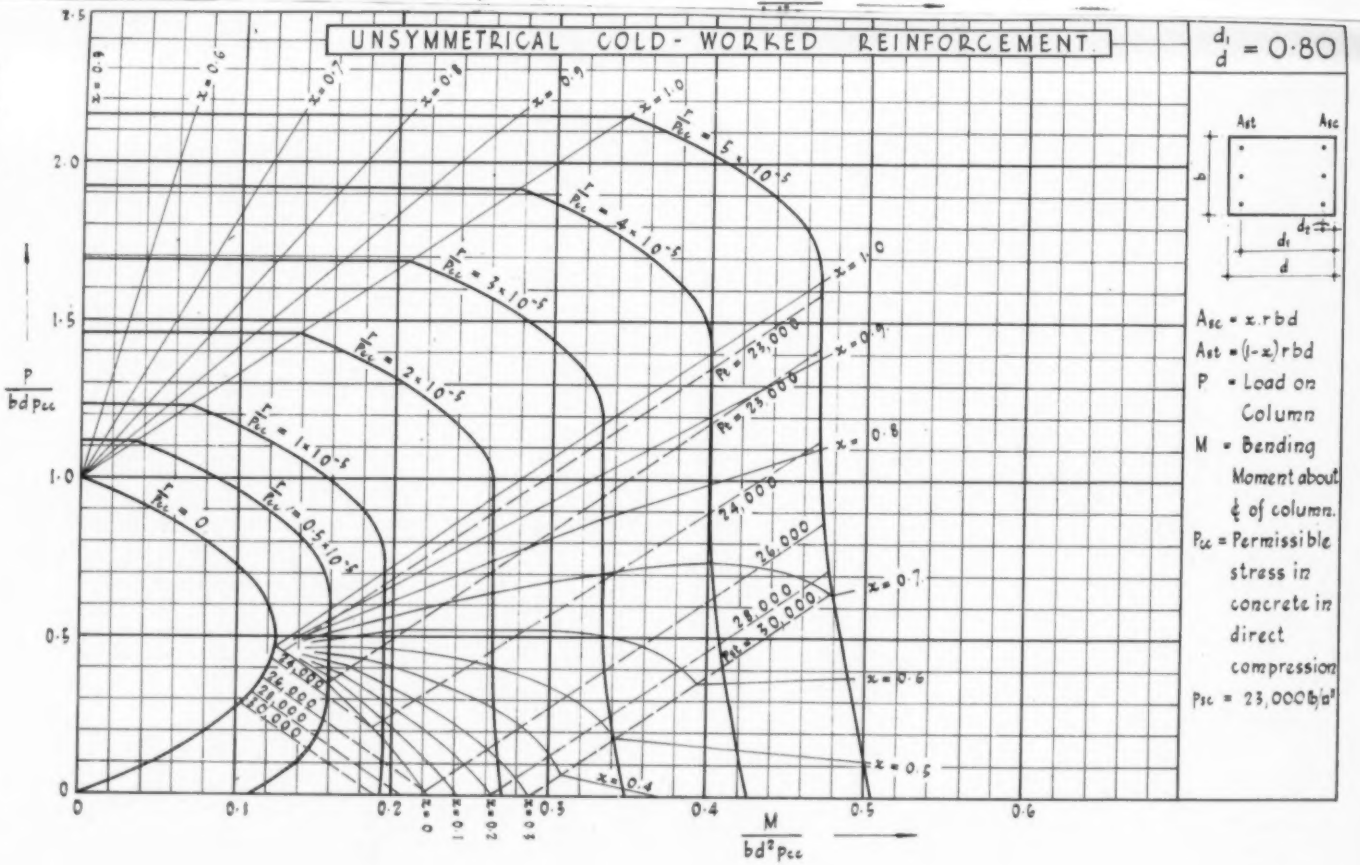
Therefore $n = \dfrac{100{,}000}{100{,}000 + \dfrac{60p_{st} \times 10^6}{E_s}}$. When $p_{st} = 30{,}000$ lb. per square inch, $E_s = 13{\cdot}33 \times 10^6$ lb. per square inch and $n = 0{\cdot}426$. For values of $n$ less than 0·426 the section will fail in tension and for values greater than 0·426 it will fail in compression.

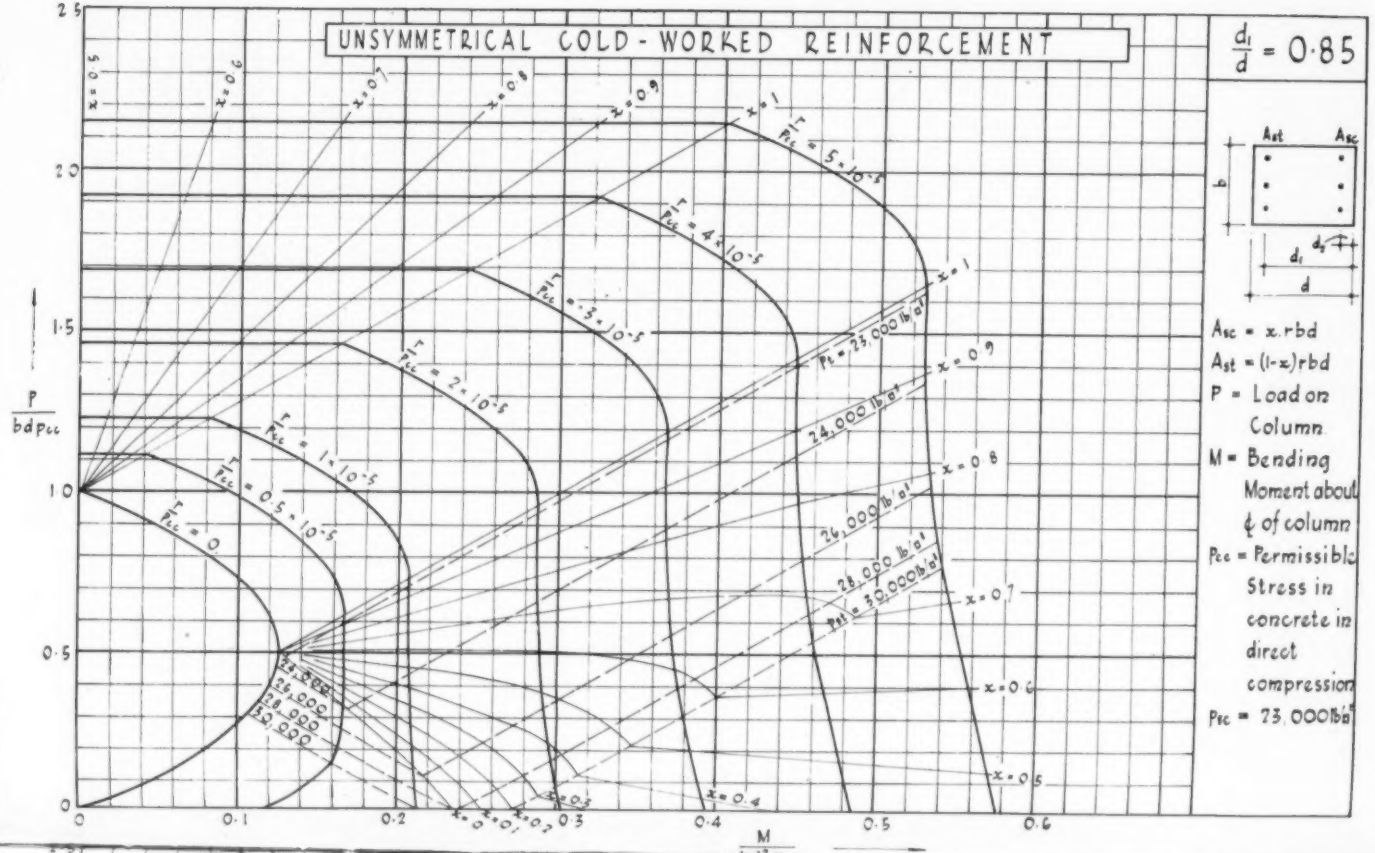
### Failure in Compression.

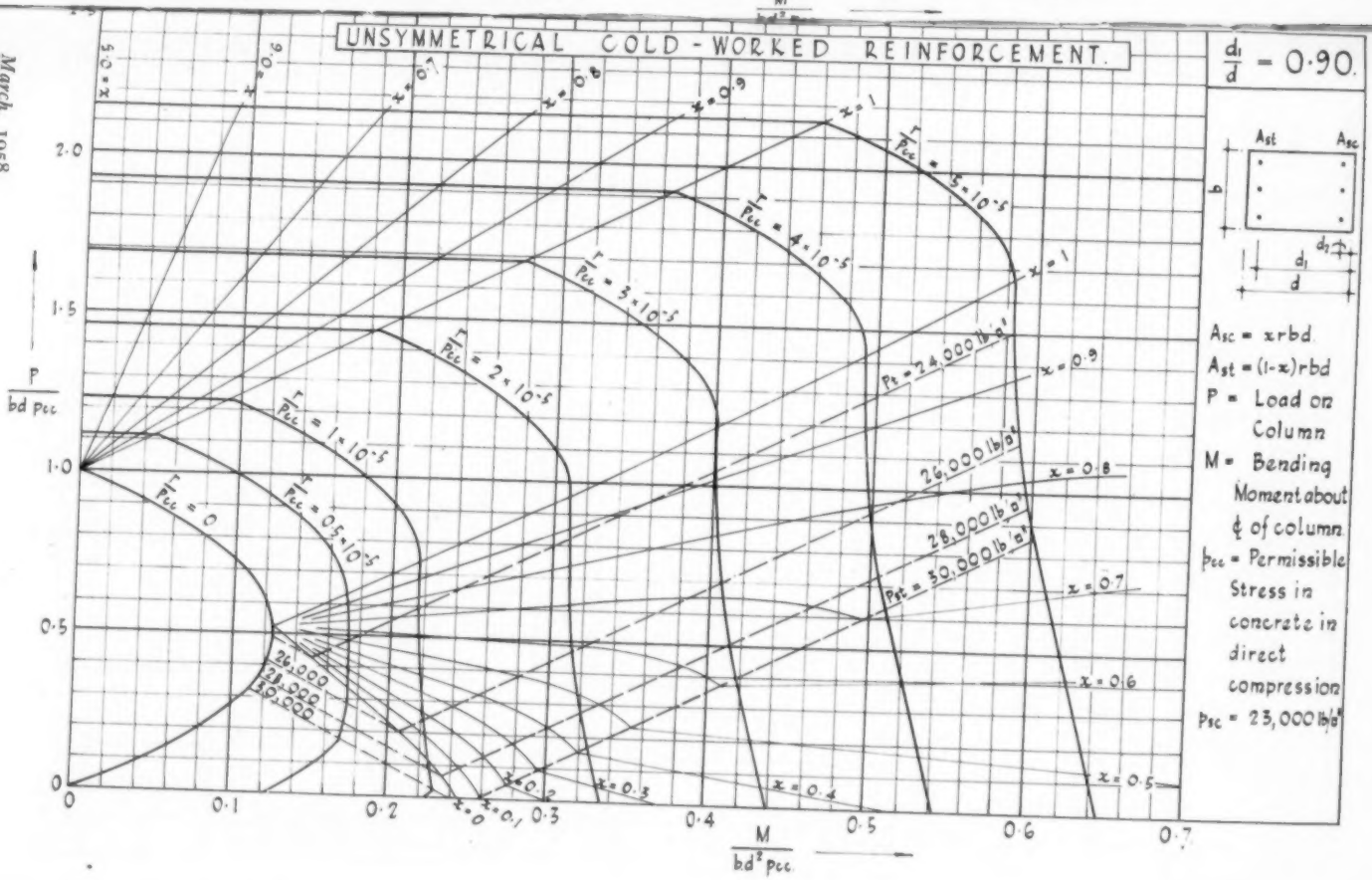
The strain in the tensile steel at ultimate load is $\dfrac{1 - n}{300n}$, and *Graph* 1 (November, 1957, p. 362) represents the stress-strain relationship for cold-worked bars with a minimum yield stress of 60,000 lb. per square inch. If the stress is greater than 46,000 lb. per square inch, it may be represented by the upper straight line

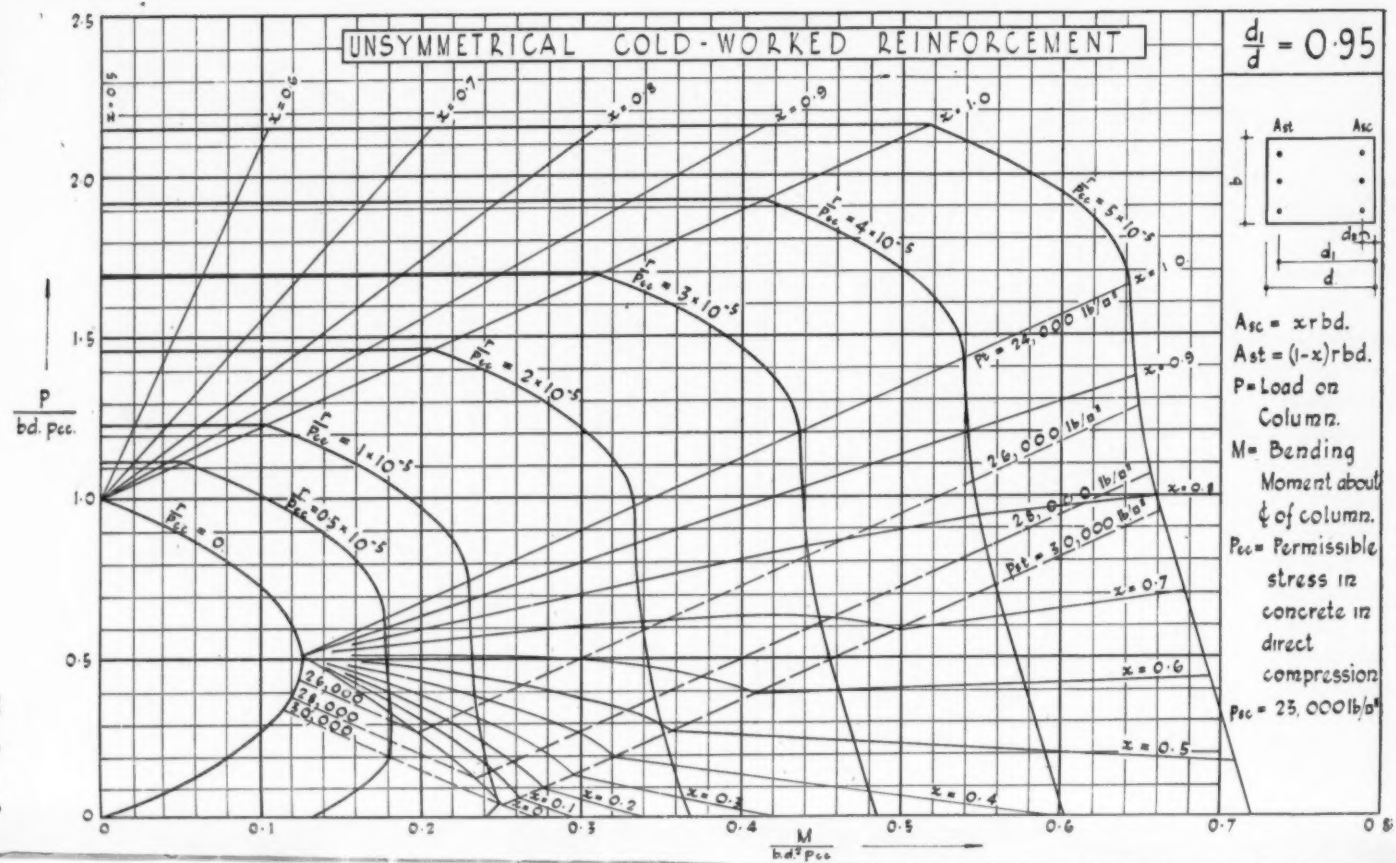
* Previous articles in this series appeared in this Journal for November and December, 1957.


UNSYMMETRICAL COLD-WORKED REINFORCEMENT
d1/d = 0.75
P / bd pcc
M / bd²pcc
x = 1.0
x = 0.9
x = 0.8
x = 0.7
x = 0.6
x = 0.5
r/pcc = 5×10⁻⁵
r/pcc = 4×10⁻⁵
r/pcc = 3×10⁻⁵
r/pcc = 2×10⁻⁵
r/pcc = 1×10⁻⁵
r/pcc = 0.5×10⁻⁵
r/pcc = 0
fs = 30,000 lb/in²
23,000
24,000 lb/in²
Ast
Asc
b
d1
d2
d
Asc = x.rbd
Ast = (1-x)rbd
P = Load on column
M = Bending Moment about ℄ of column
pcc = Permissible stress in concrete in direct compression.
psc = 23,000 lb/in²


UNSYMMETRICAL COLD-WORKED REINFORCEMENT
d1/d = 0·80
P / bd pcc
M / bd² pcc
x = 0·0
x = 0·1
x = 0·2
x = 0·3
x = 0·4
x = 0·5
x = 0·6
x = 0·7
x = 0·8
x = 0·9
x = 1·0
r/Pcc = 5×10⁻⁵
r/Pcc = 4×10⁻⁵
r/Pcc = 3×10⁻⁵
r/Pcc = 2×10⁻⁵
r/Pcc = 1×10⁻⁵
r/Pcc = 0·5×10⁻⁵
r/Pcc = 0
Pt = 23,000
Pt = 24,000
Pt = 26,000
Pt = 28,000
Pt = 30,000
Asc = x.rbd
Ast = (1-x)rbd
P = Load on Column
M = Bending Moment about ℄ of column.
Pcc = Permissible stress in concrete in direct compression
Psc = 23,000 lb/in²


UNSYMMETRICAL COLD-WORKED REINFORCEMENT
d1/d = 0·85
Ast Asc
d
d1
d2
Asc = x.rbd
Ast = (1-x)rbd
P = Load on Column
M = Bending Moment about ℄ of column
Pcc = Permissible Stress in concrete in direct compression
Pcc = 73,000 lb/in²
P / bdPcc
M / bd²Pcc
x = 0·0
x = 0·1
x = 0·2
x = 0·3
x = 0·4
x = 0·5
x = 0·6
x = 0·7
x = 0·8
x = 0·9
x = 1
T/Pcc = 5×10⁻⁵
T/Pcc = 4×10⁻⁵
T/Pcc = 3×10⁻⁵
T/Pcc = 2×10⁻⁵
T/Pcc = 1×10⁻⁵
T/Pcc = 0·5×10⁻⁵
T/Pcc = 0
Pst = 23,000 lb/in²
24,000 lb/in²
26,000 lb/in²
28,000 lb/in²
Pst = 30,000 lb/in²
23,000
24,000
26,000
28,000
30,000


UNSYMMETRICAL COLD-WORKED REINFORCEMENT.
$\frac{d_1}{d} = 0.90$.
$\frac{P}{bd\,p_{cc}}$
$\frac{M}{bd^2 p_{cc}}$
$A_{sc} = xrbd$
$A_{st} = (1-x)rbd$
P = Load on Column
M = Bending Moment about ℄ of column
$p_{cc}$ = Permissible Stress in concrete in direct compression
$p_{sc}$ = 23,000 lb/in²
$\frac{r}{p_{cc}} = 5\times10^{-5}$
$\frac{r}{p_{cc}} = 4\times10^{-5}$
$\frac{r}{p_{cc}} = 3\times10^{-5}$
$\frac{r}{p_{cc}} = 2\times10^{-5}$
$\frac{r}{p_{cc}} = 1\times10^{-5}$
$\frac{r}{p_{cc}} = 0.5\times10^{-5}$
$\frac{r}{p_{cc}} = 0$
$p_{st}$ = 24,000 lb/in²
$p_{st}$ = 26,000 lb/in²
$p_{st}$ = 28,000 lb/in²
$p_{st}$ = 30,000 lb/in²
x = 0.9
x = 0.8
x = 0.7
x = 0.6
x = 0.5
x = 0.4
x = 0.3
x = 0.2
x = 0.1
x = 0


UNSYMMETRICAL COLD-WORKED REINFORCEMENT
d1/d = 0.95
Ast
Asc
d
d1
Asc = xrbd.
Ast = (1-x)rbd.
P = Load on Column.
M = Bending Moment about ℄ of column.
Pcc = Permissible stress in concrete in direct compression
Pcc = 23,000 lb/in²
P / bd. Pcc
M / bd² Pcc
x = 1.0
x = 0.9
x = 0.8
x = 0.7
x = 0.6
x = 0.5
x = 0.4
x = 0.3
x = 0.2
x = 0.1
x = 0
r Pst = 0
r Pst = 0·5 × 10⁻⁵
r Pst = 1 × 10⁻⁵
r Pst = 2 × 10⁻⁵
r Pst = 3 × 10⁻⁵
r Pst = 4 × 10⁻⁵
r Pst = 5 × 10⁻⁵
Pst = 25,000 lb/in²
Pst = 26,000 lb/in²
Pst = 28,000 lb/in²
Pst = 30,000 lb/in²
24,000
28,000
30,000

on this graph, the equation of which is

$$2p_t = 38{,}800 + 4{,}720{,}000\left(\frac{1-n}{300n}\right),$$

$$\text{or} \quad p_t = 11{,}550 + \frac{7850}{n} \text{ lb. per square inch} \quad . \quad . \quad (1)$$

If the stress is less than 46,000 lb. per square inch at ultimate load the lower straight line can be used. This corresponds to the normal stress-strain relationship when Young's modulus is $30 \times 10^6$ lb. per square inch, and its equation is

$$2p_t = 30{,}000{,}000\left(\frac{1-n}{300n}\right),$$

$$\text{or} \quad p_t = \frac{50{,}000}{n} - 50{,}000 \text{ lb. per square inch} \quad . \quad . \quad (2)$$

When the stress in the tensile steel is between 46,000 and 60,000 lb. per square inch equation (1) applies, and the equations of equilibrium for the section are as follows.

Resolving forces axially, $\dfrac{P}{bdp_{cc}} = 0{\cdot}85\dfrac{d_1}{d}n + 23{,}000\dfrac{r_2}{p_{cc}} - \left(11{,}550 + \dfrac{7850}{n}\right)\dfrac{r_1}{p_{cc}}$.

By using the relationship $r_1 + r_2 = r$, this can be written:

$$\frac{r_1}{p_{cc}} = \frac{0{\cdot}85\dfrac{d_1}{d}n + 23{,}000\dfrac{r}{p_{cc}} - \dfrac{P}{bdp_{cc}}}{34{,}550 + \dfrac{7850}{n}} \quad . \quad . \quad . \quad . \quad (3)$$

Calculating moments about the tensile steel,

$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 + 23{,}000\left(\frac{d_1-d_2}{d}\right)\frac{r_2}{p_{cc}} - \left(\frac{d_1-d_2}{2d}\right)\frac{P}{bdp_{cc}}$$

and by using equation (3) this can be written:

$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 + 23{,}000\left(\frac{d_1-d_2}{d}\right)\frac{r}{p_{cc}}$$

$$- 23{,}000\frac{d_1-d_2}{d}\left[\frac{0{\cdot}85\dfrac{d_1}{d}n + 23{,}000\dfrac{r}{p_{cc}} - \dfrac{P}{bdp_{cc}}}{34{,}550 + \dfrac{7850}{n}}\right] - \left(\frac{d_1-d_2}{2d}\right)\cdot\frac{P}{bdp_{cc}} \quad . \quad (4)$$

For a particular load $P$ and a given ratio of reinforcement $\dfrac{r}{p_{cc}}$ there will be three variables, $M$, $n$, and $r_1$. The relationship between $n$ and $r_1$ is given by equation (3) and this is included in equation (4), which can be differentiated with respect to $n$ to determine the value of $n$ which gives a maximum value of $M$.

$$\frac{1}{bd^2p_{cc}}\cdot\frac{\partial M}{\partial n} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 - 0{\cdot}85^2\left(\frac{d_1}{d}\right)^2 n$$

$$-23{,}000\frac{d_1-d_2}{d}\left\{\frac{0{\cdot}85\dfrac{d_1}{d}\left(34{,}550+\dfrac{7850}{n}\right)+\dfrac{7350}{n^2}\left(0{\cdot}85\dfrac{d_1}{d}n+23{,}000\dfrac{r}{p_{cc}}-\dfrac{P}{bdp_{cc}}\right)}{\left(34{,}550+\dfrac{7850}{n}\right)^2}\right\}=0,$$

which simplifies to

$$862{\cdot}45\left(\frac{d_1}{d}\right)^2 n^3 + \left[728{\cdot}28\left(\frac{d_1}{d}\right)^2 - 675{\cdot}51\frac{d_1}{d}\right]n^2 + \left[197{\cdot}27\left(\frac{d_1}{d}\right)^2 - 306{\cdot}91\frac{d_1}{d}\right]n$$
$$- 52{\cdot}38\left(\frac{d_1}{d}\right)^2 + 180{\cdot}55\left(2\frac{d_1}{d} - 1\right)\left(23{,}000\frac{r}{p_{cc}} - \frac{P}{bdp_{cc}}\right) = 0 \quad . \quad . \quad (5)$$

Equation (5) is valid only for values of $n$ which when substituted in (1) give values of $p_t$ between 23,000 and 30,000, and when substituted in (3) give values of $r_1$ between 0 and $r$. As $n$ increases so $p_t$ and $r_1$ decrease, and if $p_t$ decreases to 23,000 before $r_1$ decreases to zero (this occurs only if $\frac{d_1}{d} = 0{\cdot}75$ or $0{\cdot}80$), then equations (4) must be rewritten using the tensile stress in the steel given by equation (2). Substitution and differentiation then lead to another cubic equation in $n$ for a maximum value of $M$.

When $\frac{d_1}{d} = 0{\cdot}90$, equation (5) reduces to

$$698{\cdot}58n^3 - 18{\cdot}05n^2 - 116{\cdot}43n - 42{\cdot}43 + 144{\cdot}44\left(23{,}000\frac{r}{p_{cc}} - \frac{P}{bdp_{cc}}\right) = 0.$$

By plotting $n$ against $698{\cdot}58n^3 - 18{\cdot}05n^2 - 116{\cdot}43n$ it is possible to determine the value of $\left\{-42{\cdot}43 + 144{\cdot}44\left(23{,}000\frac{r}{p_{cc}} - \frac{P}{bdp_{cc}}\right)\right\}$ for any particular value of $\frac{r}{p_{cc}}$ and $\frac{P}{bdp_{cc}}$ and then read off the corresponding value of $n$ from the graph. $\frac{M}{bd^2p_{cc}}$ can then be calculated from equation (4), $r_1$ from equation (3), and $p_t$ from equation (1).

When $r_1 = 0$ all the reinforcement is at the compressive face and equation (4) no longer applies. Rewriting the equations of equilibrium to allow for the absence of tensile steel, (6) and (7) are obtained.

$$\frac{P}{bdp_{cc}} = 0{\cdot}85\frac{d_1}{d}.n + 23{,}000\frac{r}{p_{cc}} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 + 23{,}000\left(\frac{d_1 - d_2}{d}\right)\frac{r}{p_{cc}} - \left(\frac{d_1 - d_2}{2d}\right)\frac{P}{bdp_{cc}} \quad . \quad (7)$$

Equations (6) and (7) are valid until $n = \frac{1}{0{\cdot}85\frac{d_1}{d}}$, when all the concrete is in compression and the bending moment is resisted by the total area of steel at the compressive face. This is the greatest value of $\frac{P}{bdp_{cc}}$ which can be obtained, and for smaller bending moments the distribution of the compressive steel must be more uniform, as given by (8) and (9).

Resolving forces axially, $\frac{P}{bdp_{cc}} = 1 + 23{,}000\frac{r}{p_{cc}}$. . . . . (8)

Calculating moments about the axis of the column,

$$\frac{M}{bd^2p_{cc}} = 23{,}000\left(\frac{r_2}{p_{cc}} - \frac{r_1}{p_{cc}}\right)\left(\frac{d_1 - d_2}{2d}\right). \quad . \quad . \quad . \quad (9)$$

When the value of $n$ obtained from (5) is 0·426, then $p_t = 30{,}000$ lb. per square inch and the lower limit of applicability of this equation is reached. Thus the stress in the tensile steel is 30,000 lb. per square inch and the stress in the compressive steel is 23,000 lb. per square inch, and the equations of equilibrium become

$$\frac{P}{bdp_{cc}} = 0{\cdot}85\frac{d_1}{d}n + 23{,}000\frac{r_2}{p_{cc}} - 30{,}000\frac{r_1}{p_{cc}} \quad . \quad . \quad . \quad (10)$$

and

$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 + 23{,}000\left(\frac{d_1 - d_2}{d}\right)\frac{r_2}{p_{cc}} - \left(\frac{d_1 - d_2}{2d}\right)\frac{P}{bdp_{cc}}. \quad . \quad (11)$$

For a given value of $\frac{P}{bdp_{cc}}$, $\frac{M}{bd^2p_{cc}}$ in equation (11) is a maximum when $n$ is a maximum, and the maximum possible value of $n$ is 0·426. Thus equations (10) and (11) represent a region of balanced failure and with $n = 0{\cdot}426$ can be rewritten:

$$\frac{r_1}{p_{cc}} = \frac{0{\cdot}3621\frac{d_1}{d} + 23{,}000\frac{r}{p_{cc}} - \frac{P}{bdp_{cc}}}{53{,}000} \quad . \quad . \quad . \quad (10a)$$

and

$$\frac{M}{bd^2p_{cc}} = 0{\cdot}3621\left(\frac{d_1}{d}\right)^2 - 0{\cdot}0656\left(\frac{d_1}{d}\right)^2 + 23{,}000\left(\frac{d_1 - d_2}{d}\right)\frac{r_2}{p_{cc}} - \left(\frac{d_1 - d_2}{2d}\right)\frac{P}{bdp_{cc}} \quad . \quad (11a)$$

Using (10a) and (11a), $\frac{P}{bdp_{cc}}$ can be plotted against $\frac{M}{bd^2p_{cc}}$ for fixed values of $\frac{r}{p_{cc}}$ and varying proportions of $r_1$ and $r_2$.

When $r_1 = r$ in equation (3) all the reinforcement is at the tensile face and equations (3) and (4) no longer apply. The stress in the tensile steel is still less than 30,000 lb. per square inch, however, and there is therefore a region of compressive failure when all the steel is at the tensile face. This region is bounded by the lines $x = 0$ and $p_{st} = 30{,}000$ lb. per square inch, and the equations of equilibrium are

$$\frac{P}{bdp_{cc}} = 0{\cdot}85\frac{d_1}{d}n - \left(11{,}550 + \frac{7850}{n}\right)\frac{r}{p_{cc}} \quad . \quad . \quad . \quad (12)$$

and

$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 - \left(\frac{d_1 - d_2}{2d}\right)\frac{P}{bdp_{cc}} \quad . \quad . \quad (13)$$

These are valid until $n$ becomes 0·426, when tensile failure occurs.

## Tensile Failure.

When $n$ is less than 0·426 failure is caused by the yielding of the tensile steel, and for a given load $P$ the maximum value of $M$ is obtained when all the steel is in tension. The equations of equilibrium are then

$$\frac{P}{bdp_{cc}} = 0{\cdot}85\frac{d_1}{d}n - 30{,}000\frac{r}{p_{cc}} \quad . \quad . \quad . \quad . \quad (14)$$

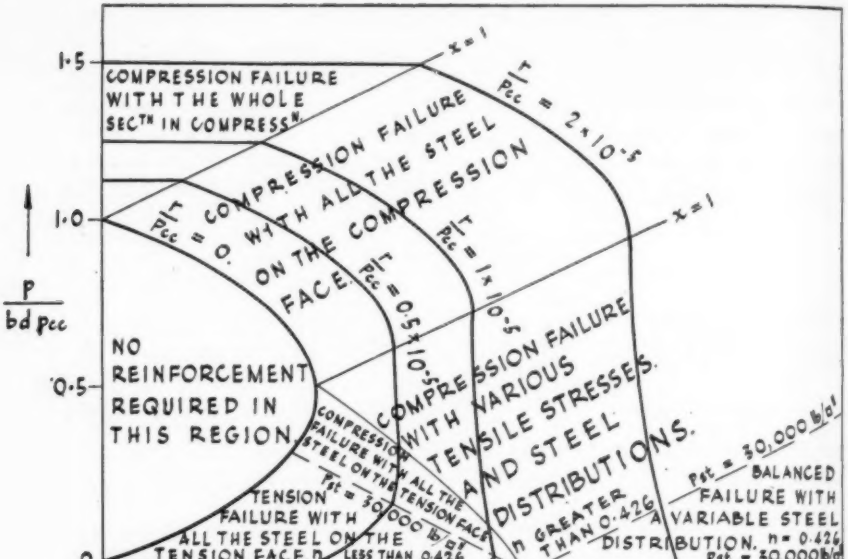

1·5
1·0
0·5
0
P / bd pcc
COMPRESSION FAILURE WITH THE WHOLE SECTN IN COMPRESSN.
x = 1
f/f'cc = 2 × 10⁻⁵
f/f'cc = COMPRESSION FAILURE WITH ALL THE STEEL ON THE COMPRESSION FACE. 0
f/f'cc = 0·5 × 10⁻⁵
f/f'cc = 1 × 10⁻⁵
x = 1
NO REINFORCEMENT REQUIRED IN THIS REGION
COMPRESSION FAILURE WITH VARIOUS TENSILE STRESSES AND STEEL DISTRIBUTIONS.
COMPRESSION FAILURE WITH ALL THE STEEL ON THE TENSION FACE. fst = 30,000 lb/sq in
TENSION FAILURE WITH ALL THE STEEL ON THE TENSION FACE. fst = 30,000 lb/sq in LESS THAN 0·426
fst = 30,000 lb/sq in BALANCED FAILURE WITH A VARIABLE STEEL DISTRIBUTION. n = 0·426 fst = 30,000 lb/sq in
n GREATER THAN 0·426
0
0·1
M / bd² pcc
0·2
x / d = 0
0·3
0·4

Fig. 1.

and
$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 - \left(\frac{d_1 - d_2}{2d}\right)\frac{P}{bdp_{cc}} \quad . \quad . \quad (15)$$

From these equations $\frac{P}{bdp_{cc}}$ can be plotted against $\frac{M}{bd^2p_{cc}}$ as before.

*Fig.* 1 illustrates the range of each of the various types of failure; it is not necessary however to remember these in order to use the charts.

**Example.**—The example given in the first article is considered. A column 24 in. by 12 in. in cross section, with 1 : 2 : 4 concrete, and $p_{cc} = 760$ lb. per square inch, supports a direct load $P$ of 130,000 lb. and a bending moment $M$ of 1,300,000 in.-lb. Then

$$\frac{P}{bdp_{cc}} = \frac{130{,}000}{12 \times 24 \times 760} = 0{\cdot}595 \text{ and } \frac{M}{bd^2p_{cc}} = \frac{1{,}300{,}000}{12 \times 24^2 \times 760} = 0{\cdot}247.$$

Assuming that $\frac{d_1}{d} = 0{\cdot}90$, the graph indicates that $\frac{f}{p_{cc}} = 1{\cdot}31 \times 10^{-5}$ and $x = 0{\cdot}71$.

Therefore $A_{sc} = 0{\cdot}71 \times 1{\cdot}31 \times 10^{-5} \times 760 \times 12 \times 24 = 2{\cdot}04$ sq. in.

and $A_{st} = 0{\cdot}29 \times 1{\cdot}31 \times 10^{-5} \times 760 \times 12 \times 24 = 0{\cdot}84$ sq. in.

With symmetrical reinforcement the total area of steel required is 3·28 sq. in. and with unsymmetrical reinforcement the total area is 2·88 sq. in., that is a saving of 12 per cent. of the steel; when $x$ is either 1 or 0 the saving can be almost 50 per cent. of the steel required for a symmetrically-reinforced column.

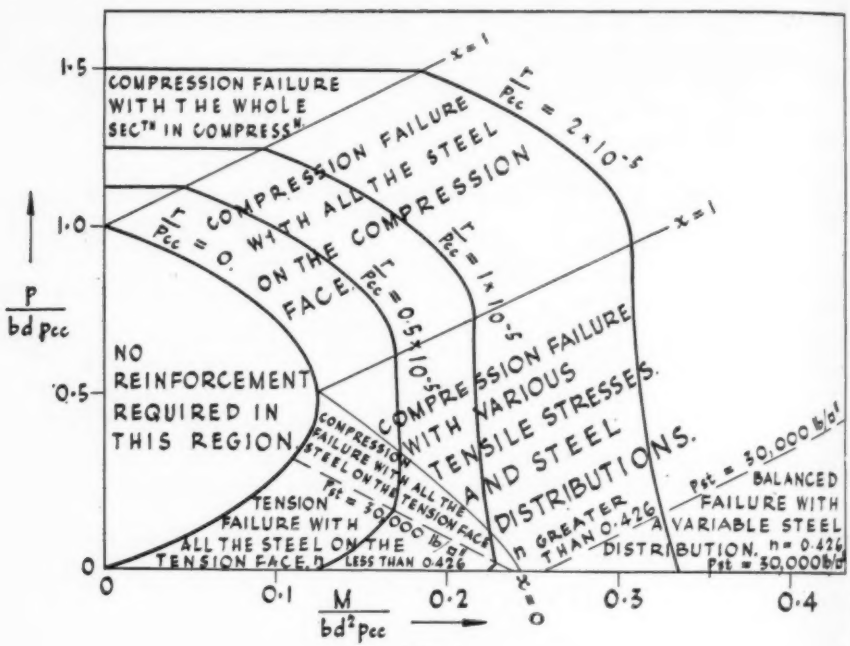
# A Parking Garage in Canada.

A PARKING garage (*Fig.* 1) which is believed to be the largest in the Commonwealth was recently completed in Vancouver. It is 323 ft. long, 131 ft. wide, and 100 ft. high from the basement to the top of the elevator, has seven floors and a basement, and can accommodate 870 cars; provision was made for the addition of another floor if required. The design and drawings were completed in 3½ months.

Walls are omitted in order to reduce cost, to allow the entry of as much daylight as possible, and so that exhaust gases will be removed by natural ventilation. The floors are designed as balanced cantilevers (*Fig.* 2), the weight being reduced by tapering the cantilevers and using ribbed slabs for the central span. The slabs are 3 in. thick, including an allowance of 1 in. for wear, and the ribs are 4 in. wide. The end bays of the structure are shortened to permit a constant depth of slab. The floors are designed to resist a live load of 75 lb. per square foot and an alternative concentrated load of 2000 lb. in any position. Enclosed smoke-proof load-bearing stair-wells provide some resistance to forces due to wind and earthquake, and additional resistance is provided by the spiral floors and the end walls, which form vertical trusses; because of this, most of the bending stresses in the columns (which are small compared with the axial stresses) are caused by live loads, shrinkage, and effects of temperature. No expansion or contraction joints are provided; stresses due to shrinkage were allowed for in the design and no cracking of the 3-in. roof slab has occurred. The open nature of the building allows the entry of large quantities of rainwater, and the floors are sloped to remove this rapidly without inconvenience to pedestrians.

The arrangement of the garage is symmetrical, with a double ramp on the longitudinal centre-line. Each floor above the second is alike, the finished surface being 10 ft. above the floor below. The lowest point of each floor is in the centre of the building. A circular ramp, extending through 90 deg. and leading to the

**Fig. 1.**


1·5
1·0
0·5
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P / bd pcc
COMPRESSION FAILURE WITH THE WHOLE SECTN IN COMPRESSN.
x = 1
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COMPRESSION FAILURE WITH ALL THE STEEL ON THE COMPRESSION FACE
p/f pcc = 0
p/f pcc = 0·5 x 10·5
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COMPRESSION FAILURE WITH ALL THE STEEL ON THE TENSION FACE
TENSION FAILURE WITH ALL THE STEEL ON THE TENSION FACE. n LESS THAN 0·426
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fst = 30,000 lb/sq in
fst = 30,000 lb/sq in
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0·1
M / bd² pcc
0·2
x = 0
0·3
0·4

Fig. 1.

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$$\frac{M}{bd^2p_{cc}} = 0{\cdot}85\left(\frac{d_1}{d}\right)^2 n - \frac{0{\cdot}85^2}{2}\left(\frac{d_1}{d}\right)^2 n^2 - \left(\frac{d_1 - d_2}{2d}\right)\frac{P}{bdp_{cc}} \quad . \quad . \quad (15)$$

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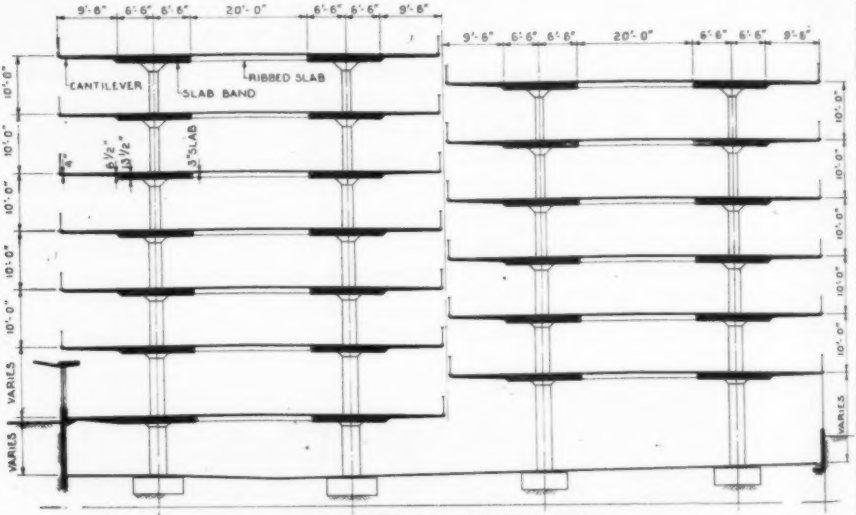
The arrangement of the garage is symmetrical, with a double ramp on the longitudinal centre-line. Each floor above the second is alike, the finished surface being 10 ft. above the floor below.' The lowest point of each floor is in the centre of the building. A circular ramp, extending through 90 deg. and leading to the

**Fig. 1.**

back section, has a rise of about 2 ft. The floor has a uniform gradient of 1·7 per cent. for 112 ft. and then turns to the right. A circular ramp of 180 deg. around the fire staircase gains a further 2 ft. and leads to the front section, facing the centre of the garage. A further 1·7 per cent. gradient gains 2 ft. in 112 ft., and the final 2 ft. are gained by a right-hand turn along a circular ramp of 90 deg. which leads to the lowest point on the next floor, 10 ft. above the starting point (*Fig.* 3). The back and front sections of each floor are joined only at the ramps at each end and at the centre; elsewhere handrails are provided on each side of a 12-in. gap. The circular ramps have a double super-elevation. With very few exceptions, no part of the floors is level.

Structurally the building is symmetrical about both centre-lines. There are four rows of circular columns spaced at 28-ft.




CROSS SECTION THROUGH GARAGE

**Fig. 2.**

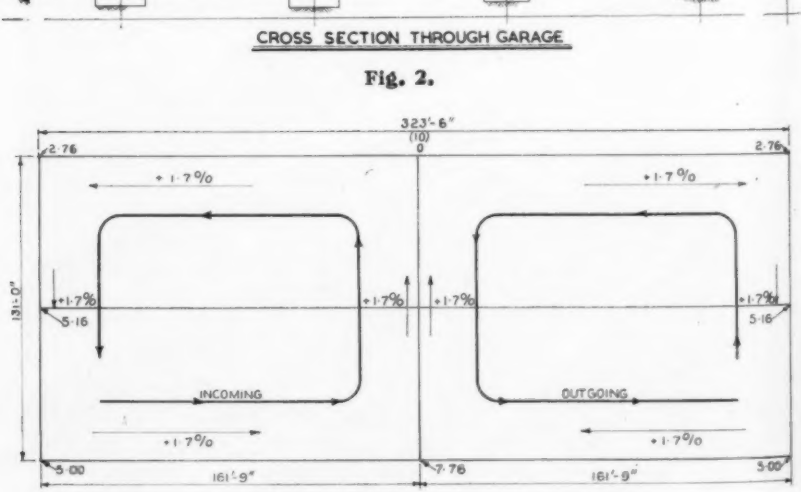



**Fig. 3.—Plan of Ramp.**

**Fig. 4.**

centres longitudinally and at 33-ft. centres laterally, the outer rows being 16 ft. from the faces of the building. Slab-bands 13½ in. deep and 13 ft. wide span longitudinally between the columns, and the beam-and-slab section spans 20 ft. between the bands. The cantilevers extend 9 ft. 6 in. towards the outer edges of the building and towards the centre-line, those towards the centre-line being separated by the 12-in. gap previously mentioned. The cantilevers vary in depth from 4 in. at the outer edge to 6½ in. where they join the slab-band. At the end ramp a 5-in. slab joins the bands, and at the central ramp a further beam-and-slab section spans between the bands. All the ribs are 10 in. deep, and are at 2-ft. centres.

The gable walls are 10 in. thick at the slab-bands and 6 in. elsewhere. The circular columns vary in diameter from 36 in. in the basement to 18 in. under the seventh floor.

The parking arrangement is designed to cater for drivers who find difficulty in parking a car. The stalls are 8 ft. 4 in. wide, and are angled to allow entry and departure with a minimum of manœuvring; the width of the driveway is about 26 ft. (*Fig.* 4). The garage is arranged for one-way traffic, with only left-hand turns and no cross-traffic. Peak traffic is expected to occur when the shops close; one entrance and two exits are therefore provided.

The garage was built in seven months; each floor, which has an area of 44,000 sq. ft. and contains 1100 cu. yd. of concrete, was built in ten working days. Adjustable steel props were used to support the shuttering, which consisted of plywood supported on timber stringers; stiffened cardboard boxes, coated with bitumen, were used to form the ribs of the transverse beams (*Fig.* 5). Ready-mixed concrete was used, the strength of which was specified to be 3000 lb. per square inch at 28 days. A total of 10,000 cu. yd. of concrete, 1200 tons of reinforcing steel, and 300,000 sq. ft. of plywood shuttering

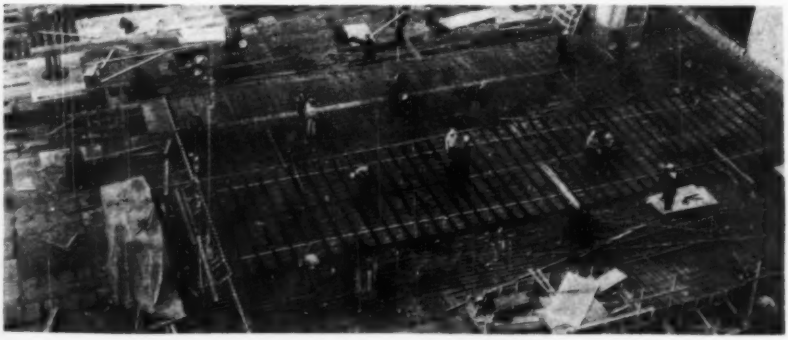


**Fig. 5.—Cardboard Boxes to form Ribs in Floor.**

was used. A steel bridge of 75 ft. span, which connects the garage with a department store, was erected, and the work of fifteen sub-contractors completed, in two weeks. The value of the contract was $1,250,000.

The owners of the garage are Woodward Department Stores, Ltd. The consulting engineers were Messrs. Robertson, Kolbein, Teevan, and Gallaher, and the main contractors were Messrs. John Laing & Son (Canada), Ltd.

## The Education of Engineers.

We have received the following from Mr. H. R. Shortall, B.A., B.A.I., A.M.I.C.E., of Northern Rhodesia.

I have just read in your November 1957 number a reprint of an address on the above subject and your Editorial comment.

I wish to bring to your notice and that of the author of the address the course of study for an engineering degree at Trinity College, Dublin University. At this college, when the full course in Engineering has been completed and the necessary examination passed, a degree in engineering is not conferred unless the candidate also possesses a degree in arts. If this is not the case, only a diploma in engineering is awarded.

The four-years' course of study in arts is taken concurrently with that in engineering. During the first two years of the arts course the subjects studied include Latin (compulsory), a modern language, and logic. The student attends lectures and is examined in the subjects chosen. During the second two years of the arts course the engineering student attends lectures and is examined in two of the following subjects: geography, astronomy, and political economy. All interim examinations and the degree examination include essays on a subject set from one of two English classics, different books being chosen for each examination, or a free subject.

Subjects studied in the arts course by an engineering student must not include any which are already in his engineering curriculum. The object of this stipulation, and of the arts course itself, is to ensure that a graduate in engineering possesses not only a knowledge of engineering but a knowledge of other subjects, and is not half-educated. In fact he has received a liberal education. By compelling students to take a degree in arts before a degree in engineering can be obtained the University maintains its status as a seat of learning and culture, and prevents it from becoming solely a dispenser of technical knowledge. I might add that this stipulation regarding the possession of an arts degree applies to all technological degree courses at Trinity College, Dublin.

How long these regulations have been in force at Trinity with regard to engineering students I cannot say with any certainty without recourse to records, but they have existed for at least thirty years if not since the establishment of the Engineering Faculty in the late nineteenth century.

[We have previously pointed out that at Dublin University a Pass Degree in Arts is required before a candidate can obtain a degree in engineering. In November 1950 we reported that some U.S.A. technological institutes had decided not to accept students for engineering courses unless they had completed three years' study of the liberal arts at other colleges; the arts courses emphasise the basic sciences and mathematics and are followed by two years' specialised study at the engineering college.

In this journal for January 1950 we wrote: " If we require only technologists then we can dispense with the arts and continue to produce men who think that computing stresses is engineering and that routine work in a laboratory is science. But a really great engineer or scientist must have a liberal education that puts at his disposal all natural knowledge."—Editor.]

# A Reservoir with Plain Concrete Walls.

## CONSTRUCTED IN SEVEN MONTHS.

A SERVICE reservoir with a capacity of 2,500,000 gallons constructed for the Urban District Councils of Paignton and Brixham is in two parts; the part for Paignton contains 1,500,000 gallons, and that for Brixham 1,000,000 gallons. It is equipped with a pump-house, a meter-house, and recorder-houses, and about 1¼ miles of mains connect it to the existing mains of the two authorities. The work was completed in seven months.

*Fig.* 1 shows the reservoir during construction, and sections through the structure are given in *Fig.* 2. The internal dimensions are 255 ft. by 114 ft. The roof is supported on 65 precast columns. The walls, which are 18 ft. 9 in. high above floor level, are of plain concrete 10 ft. 6 in. thick at the base and reducing in steps to 1 ft. 6 in. at the top. A reinforced concrete counterfort wall (shown during construction in *Fig.* 3) separates the Paignton and Brixham parts, and each part is divided by a reinforced concrete wall 12 ft. high.

Two grades of concrete were used for the external walls, namely a 1 : 2 : 4 mixture (to which a waterproofer was added) for the inner 18 in., and a 1 : 3 : 5 mixture for the remainder. The two grades were placed at the same time and kept apart by a sliding shutter which was withdrawn as soon as the concrete was placed so that the two thicknesses formed a homogeneous wall. The junctions between the plain concrete walls and the reinforced concrete dividing wall were formed by incorporating the end counterforts into the plain concrete. Rubber waterstops were employed at all joints.

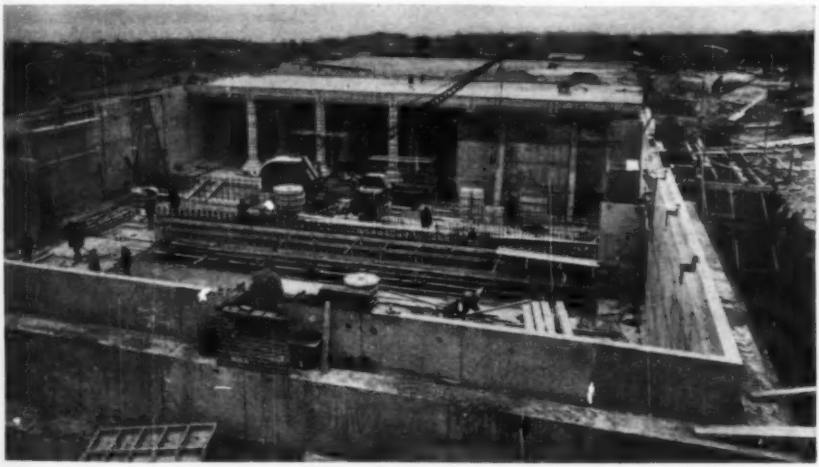


**Fig. 1.**

The floor, which is unreinforced, comprises a blinding layer 3 in. thick followed by a layer of structural concrete 6 in. thick and a top layer varying in thickness from 10 in. to 4 in. to provide drainage to the outlet and wash-out pipes. The columns and main and secondary roof beams were precast. The secondary beams form the roof, and are covered with concrete of an average thickness of 3 in. A 3-in. layer of gravel was placed on the concrete and covered with 9 in. of topsoil.

The design was influenced by the requirement that the reservoir should be finished in seven months. Plain concrete was used wherever possible because of the difficulty of obtaining steel. The

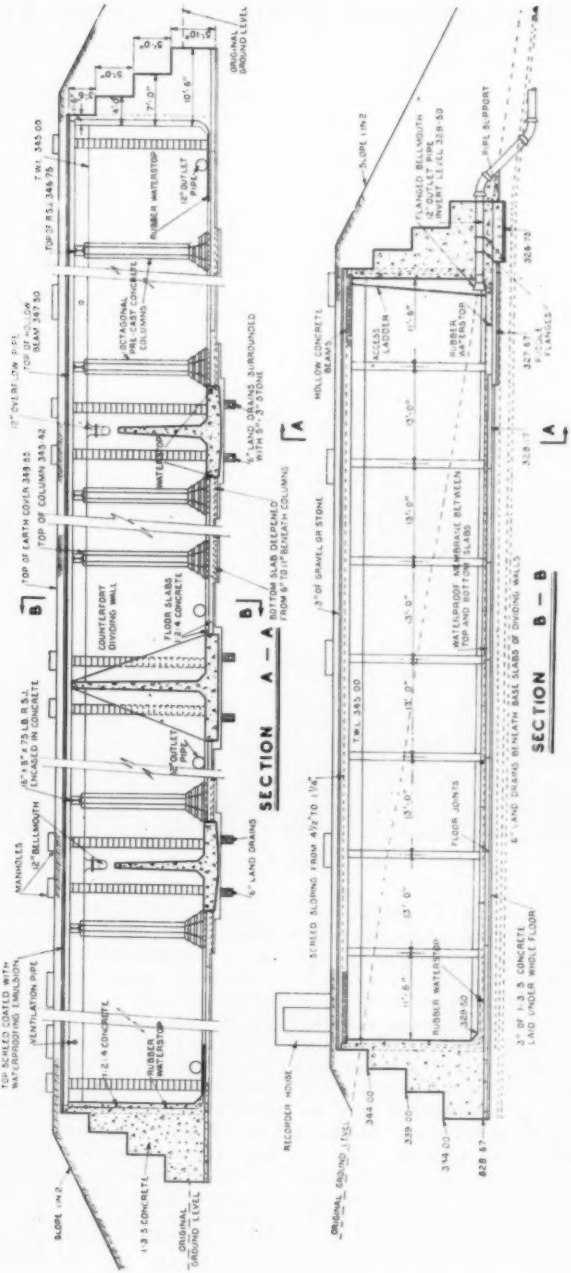

SECTION A — A
SECTION B — B
ORIGINAL GROUND LEVEL
TOP OF EARTH COVER 349.25
TOP OF COLUMN 345.42
TOP OF BEAM 347.50
TOP OF R.S.J. 346.75
TOP WL 345.00
OCTAGONAL PRE-CAST CONCRETE COLUMNS
RUBBER WATERSTOP
12" OUTLET PIPE
COUNTERFORT DIVIDING WALL
FLOOR SLABS 7" R.C. CONCRETE
BOTTOM SLAB DEEPENED FROM 6" TO 18" BENEATH COLUMNS
6" LAND DRAINS SURROUNDED WITH 3"-1" STONE
3" OF GRAVEL OR STONE
HOLLOW CONCRETE BEAMS
ACCESS LADDER
RUBBER WATERSTOP
FLANGED BELLMOUTH 12" OUTLET PIPE INVERT LEVEL 338.50
PIPE SUPPORT
SLOPE 1 IN 2
MANHOLES
12" BELLMOUTH
36" x 18" x 75 LB. R.S.J. ENCASED IN CONCRETE
SCREED SLOPING FROM 4½" TO 1½"
WATERPROOF MEMBRANE BETWEEN TOP AND BOTTOM SLABS
FLOOR JOINTS
6" LAND DRAINS BENEATH BASE SLABS OF DIVIDING WALLS
3" OF 1:3:5 CONCRETE LAID UNDER WHOLE FLOOR
TOP SCREED COATED WITH WATERPROOFING EMULSION
VENTILATION PIPE
1:2:4 CONCRETE
RUBBER WATERSTOP
1:3:5 CONCRETE
RECORDER HOUSE
344.00
339.00
334.00
328.67

Fig. 2.

excavation was mostly in shale, and was sufficiently advanced to enable the construction of the walls to be commenced in less than three weeks; 70 per cent. of the total volume of 11,500 cu. yd. was excavated during the first month. The volume of concrete cast in place was 5500 cu. yd. (an average of 60 cu. yd. per day), and 1800 tons of precast concrete were placed in position. The excavated material was used to form the banks surrounding the reservoir; it was therefore placed around the perimeter of the reservoir in the position it would occupy when the banks were formed, thereby giving the material time to consolidate.

Two tilting-drum hydraulically-operated 14/10 mixers were used. The cement was delivered loose and stored in silos. The aggregates were crushed limestone and pit sand, and drag-scrapers were used to load them into hoppers which supplied the mixers. The concrete was transported from the mixers to the shutters by dumpers, and was consolidated by petrol-driven poker vibrators. It was necessary to duplicate the mixing plant due to the use of two different

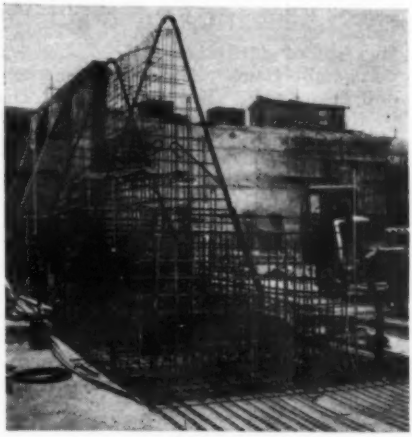


Fig. 3.

classes of concrete in the walls, but the second plant was dispensed with as soon as these were completed.

The shuttering for the walls was made of resin-bonded plywood on 6-in. by 3-in. and 4-in. by 3-in. timber frames in panels 8 ft. by 5 ft. The panels were

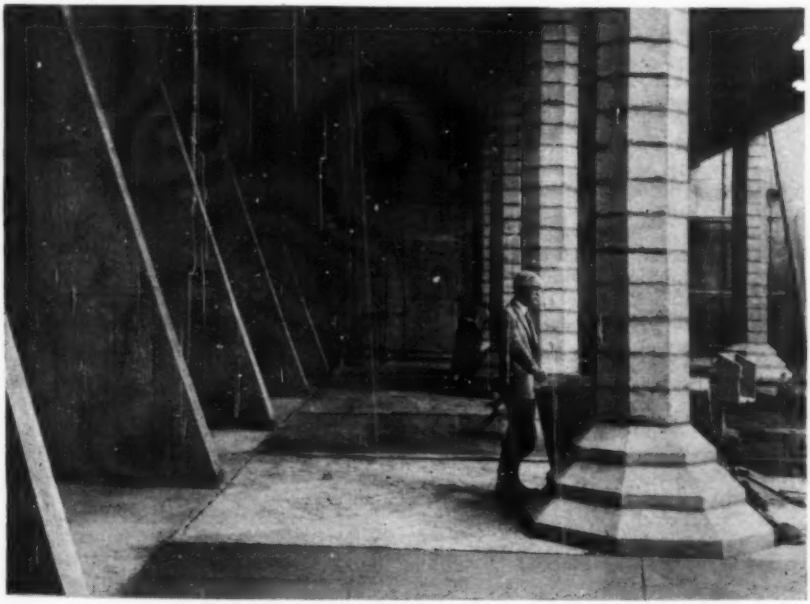


Fig. 4.

held in position by ties, which also provided the necessary fastenings for successive lifts of shuttering. Each panel was used eight times. The precast columns and beams were erected by crane. Due to the weight of the main beams (each weighs 4 tons and is 17 ft. long) a lorry-mounted crane with a 45-ft. jib was used; this crane was also used for lifting the shutter panels, placing concrete in the upper lifts of the walls with a bottom-opening skip, and placing the earth covering on top of the reservoir. The columns were cast in sections, each section being octagonal in plan with a hollow core which was filled with concrete (*Fig.* 4). The pressure allowed on the ground below the column bases is 2 tons per square foot.

The consulting engineers were Messrs. Harvey, McGill & Hayes, and the main contractors Messrs. Richard Costain, Ltd.

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### Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Nuclear Energy and Some Associated Problems, by W. L. Day. Uplands Lecture Theatre, Archery Road, St. Leonards-on-Sea. March 18. 7 p.m. And Park Hotel, Park Place, Cardiff. March 25. 7 p.m.

Powered Hand Tools, by A. F. Coare. Technical College, Wrexham. March 18. 7 p.m. Also Co-operative Society Hall, Sea View Road, Colwyn Bay. March 19. 7 p.m. And Technical College, Connah's Quay. March 20. 7 p.m.

Control of Concrete Quality on Sites, by R. A. Kenny. College of Technology, Church Street, Barnsley. March 19. 7.15 p.m. And College of Further Education, Hopwood Lane, Halifax. March 20. 7.15 p.m.

Some Aspects of Building Contracts, by N. P. Greig. Shire Hall, Bedford. March 19. 7.30 p.m.

The R.I.B.A. Form of Contract, by I. N. D. Wallace. Council Offices, Upper High Street, Guildford. March 19. 7 p.m.

Some Notable Concrete Buildings, by R. C. Blyth. Storey Institute, Lancaster. March 25. 7.30 p.m.

Safety in Excavations, by G. A. Roberts. College of Technology, Warren Street, Sheffield. March 25. 7.15 p.m.

Practical Formwork Design and Construction for Concrete, by J. G. Richardson. County Hotel, Rainsford Road, Chelmsford. March 26. 7.30 p.m.

New Traditional Methods of Building, by R. C. Scholefield. Library, Vernon Road, Scarborough. March 26. 7.15 p.m.

Avoidance of Defects in Plastering, by J. F. Ryder. Y.M.C.A., Woodside, Grosvenor Road, Skegness. March 26. 7.15 p.m.

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### A Tall Building in Venezuela.

IT is claimed that a reinforced concrete building to be erected at Caracas, Venezuela, will be the tallest in the world. It will be 614 ft. high, with 58 stories and a floor area of 1,900,000 sq. ft., and cost more than nine million dollars. The building will comprise offices and living apartments. The architect is Sr. Carlos Raul Villaneuva, of Caracas.

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### A Tall Building with Coloured Walls.

CARNEGIE HALL, New York, is to be demolished in 1959, and a 44-story office building is to be erected on the site. The new building will have vermilion-coloured walls, with gold-tinted window glazing bars and frames; the windows and wall panels will be arranged to give the structure a chequered appearance. The cost is estimated to be $22,000,000.

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### Plastic-coated Steel.

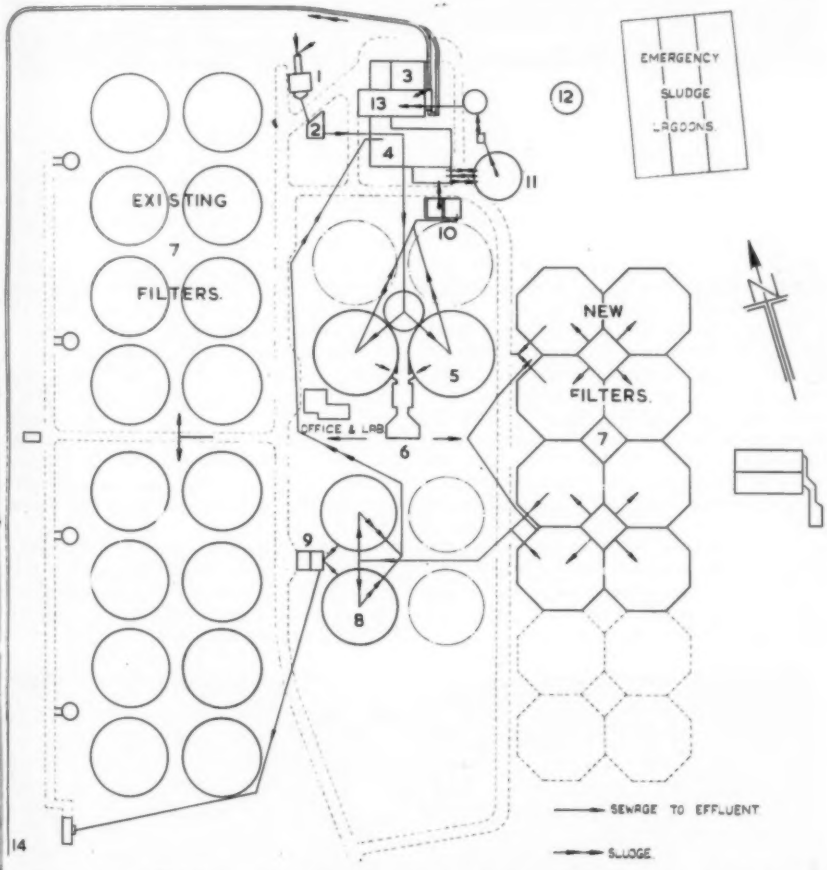
A STEEL sheet coated with plastic, produced by Messrs. John Summers & Sons, Ltd., is known as "Stelvetite". It is stated that it can be worked and treated as ordinary steel sheet, while its outer surface retains all the characteristics of P.V.C. plastic providing a permanent finish which is easy to clean and maintain. The material is produced in various colours and embossed patterns. Strip-mill cold-reduced steel is used in conjunction with the P.V.C. coating. The surface of the reverse side of the sheet can be bonderised or electro-zinc-coated.

# Sewage Disposal Works at West Bromwich.

By G. E. MARSDEN, M.I.Struct.E.

THE sewage works of the West Bromwich Corporation at Ray Hall have recently been reconstructed and extended to accommodate a dry weather flow of 4,500,000 gallons per day. *Fig.* 1 shows the arrangement of the works.

Bore-holes showed that the ground consisted of a fairly flat bed of shale of the Wenlock series which was overlain by strata of blue and brown clayey sands, some of which contained gravel; the upper layers appeared to be unreliable, and it was decided to construct the foundations in the hard shale. Free water was not encountered in the borings until the shale was entered, when the water level rose to within 4 ft. of the surface, and, as extensive excavation of the shale was




1.—Screens. 2.—Detritor. 3.—Lime mixing. 4.—Main pumphouse. 5.—Primary settlement. 6.—Filter control tower. 7.—Biological filters. 8.—Humus tanks. 9.—Recirculation pumphouse. 10.—Sludge consolidation. 11.—Sludge digestion. 12.—Gasholder. 13.—Sludge vacuum filtration. 14—Sludge tip.

**Fig. 1.—General Arrangement.**

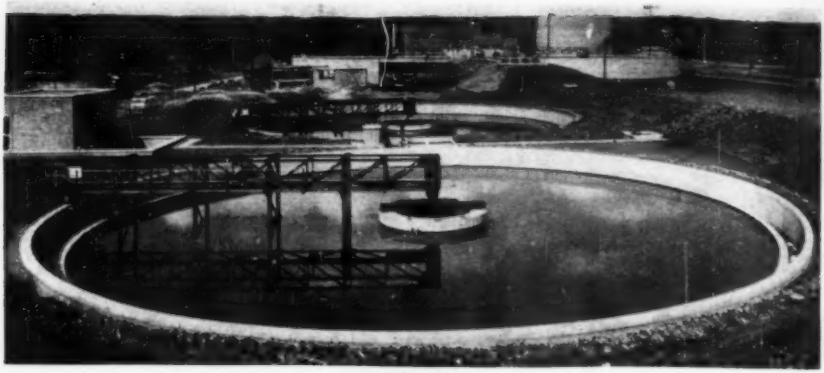


Fig. 2.

necessary, it was recommended that care be taken to prevent deterioration of the exposed shale. Tests of the ground-water near the digestion tank and the wet and dry wells of the main pumping station showed that the sulphate content exceeded 3000 parts per million, and sulphate-resisting cement was used for the concrete below ground level.

Because of the high level of the ground-water, all the tanks except the primary settlement tanks and the humus tanks were designed to prevent flotation. The operational level of the primary settlement tanks is such that their bases generally are above ground-water level, but the bases of the humus tanks are about 14 ft. below ground-water level. As these tanks were 95 ft. in diameter the provision of sufficient weight to prevent flotation would have been unreasonable; relief valves were therefore provided through the walls to safeguard them on the rare occasions when they may be empty.

The humus tanks were designed in accordance with the Code of Practice for Liquid-retaining Structures, some relaxation of these requirements being permitted for concrete required solely to prevent flotation.

Each humus tank (*Fig.* 2) has a capacity of 375,000 gallons. The depth varies from 9 ft. 6 in. at the peripheral weir to 12 ft. 9 in. at the top of the central conical sludge-hopper, which is 7 ft. 3 in. deep. The concrete baffle-box rises directly above the central sludge-hopper on six columns, each 12 in. square, joined by a 6-in. vertical concrete wall and at their tops by a beam 2 ft. wide and 1 ft. 2 in. deep which carries steel joists supporting the inner end of the scraper gear.

The main pumping station (*Fig.* 3) has a composite foundation which consists of the floor and walls of the wet and dry wells and a reinforced concrete raft supported by plain concrete piers founded on the

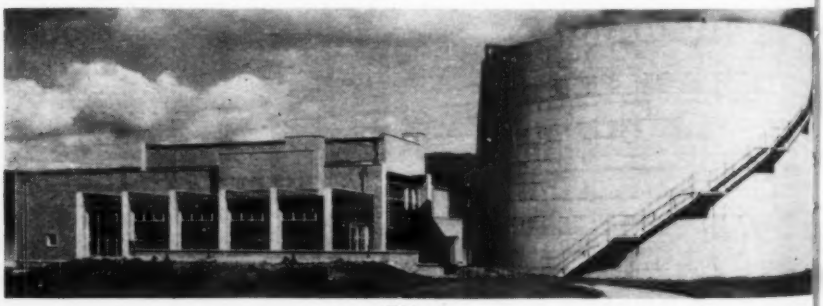


Fig. 3.

shale. The floor is 4 ft. 6 in. above the level of the ground-water and 9 ft. 6 in. above the level of the shale. The floor of the dry wells containing the pumps is 25 ft. 6 in. below the floor of the building; both the floor and the walls of the wells are 4 ft. thick. The building also contains two 165-h.p. dual-fuel engines, which are mounted on separate foundation blocks insulated from their supports and from the adjacent floor.

The sludge digestion tank (*Fig.* 4) has a capacity of 900,000 gallons. The floating steel roof may rise or fall about 8 ft. as the level of the sludge varies.

The gasholder is 45 ft. external diameter and has a capacity of 24,000 cu. ft. It is guided from a reinforced concrete pit

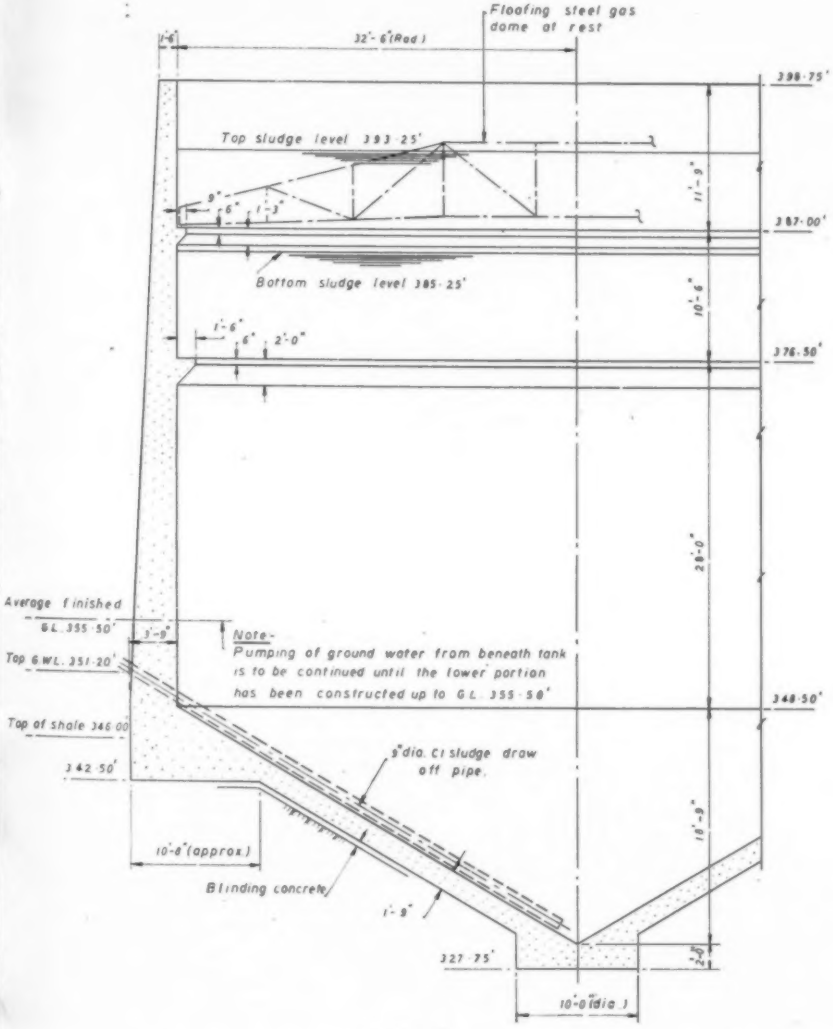
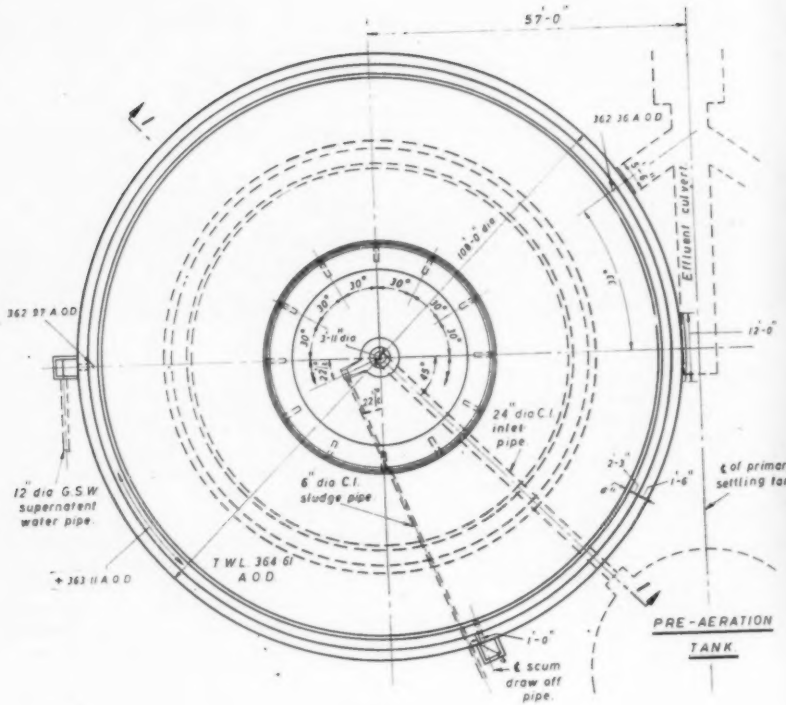
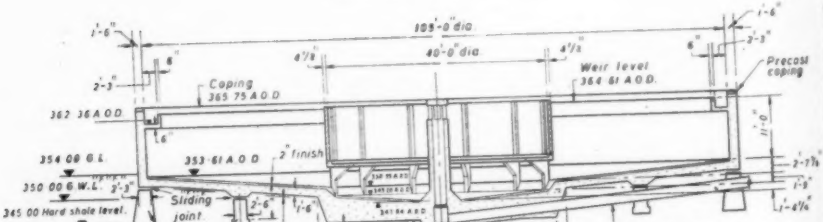



**Fig. 4.**




PLAN.




SECTION I.

**Fig. 5.**

12 ft. deep from the coping to the base. The sludge-elutriation tank has an internal diameter of 32 ft. and an average depth of 14 ft. The two sludge-consolidation tanks are 22 ft. square, with pyramidal bottoms, and have a common central wall. The coping of the vertical walls is 8 ft. above the level of the ground-water; the walls are 9 ft. high and the pyramidal bottom is 10 ft. deep.

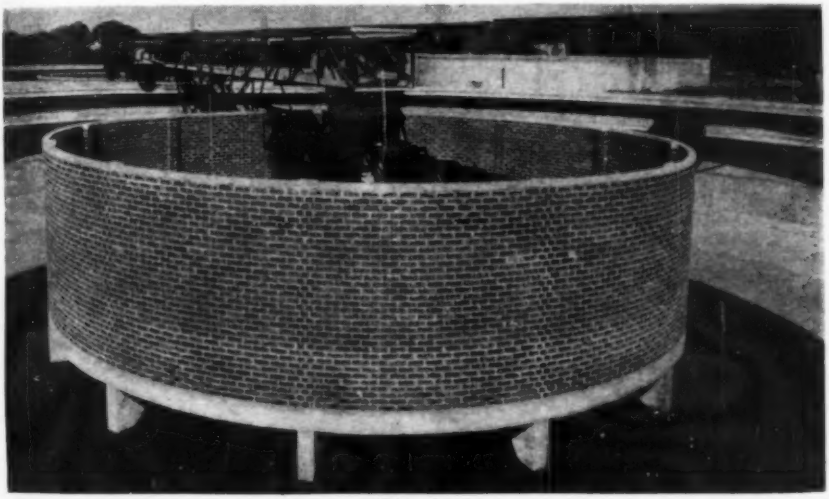


Fig. 6.

The floor of each primary settlement tank (*Figs.* 5 and 6) is supported by the sludge-collecting hopper, which is 41 ft. 3 in. diameter, and by two annular plain concrete walls from which it is separated by sliding joints. The central compartment, which contains the stirring machinery, is supported above the sludge-collecting hopper on twelve reinforced concrete columns, the heads of which are splayed inwards and connected by a ring-slab 4 ft. 9 in. wide. A small concrete beam around the outer edge of this slab supports the brick wall of the compartment. The inner end of the main scraper-mechanism and the central stirring-gear are supported by the head of the hollow central column, which is 3 ft. 11 in. diameter and contains the 24-in. main supply pipe to the tank. Each tank has a capacity of 675,000 gallons.

The recirculation pumphouse contains three 21-h.p. pumps, each of which has a capacity of 2,250,000 gallons per day. The wet and dry wells in this building are respectively 20 ft. by 13 ft. and 20 ft. by 14 ft. in plan and 13 ft. deep. The level of the ground-water is some 16 ft. above the floor of the wells, and in order to overcome flotation the external walls and floor are 2 ft. thick, with the dividing wall tapering from 1 ft. 6 in. to 1 ft. 1½ in.

Mr. H. Schofield is the Borough Engineer and Surveyor, the British Reinforced Concrete Engineering Co., Ltd., were responsible for the design of most of the reinforced concrete work, and the contractors were Messrs. R. M. Douglas (Contractors), Ltd.

# Book Reviews.

**"Elementary Theory of Structures."** By C.-K. Wang and C. L. Ecker. (New York: McGraw-Hill Book Company, Inc. 1957. Price 56*s.* 6*d.*)

THIS book includes chapters on the equilibrium of forces, shearing forces and bending moments, the analysis of lattice girders and roof trusses, influence lines, moving loads, and the analysis of statically-indeterminate beams, frames, and trusses. The methods of slope deflection and moment distribution are fully described. The American loading considered in the chapters on the analysis of bridges and roof trusses reduces the value of these sections to British readers.

The text is clearly written, and numerous diagrams, examples, and problems are included, but the exclusion of information on the theory of deflection, the theory of arches, and the analysis of frames of more than one story make it less suitable for British students than many other books already available.

**"A New Method of Recording Vibrations in the Foundation of Turbogenerators and the Structure of Power Houses."** By A. Major. (Budapest: Acta Technica. Price not stated.)

THIS paper (printed in the English language) describes the use of 26 geophones to measure the characteristics of synchronous vibrations simultaneously at a number of points. The results were analysed by determining the initial disturbances and applying Fourier's Theorem to those remaining.

**"Applied Building Construction."** By A. Medlycott. (London: Chapman & Hall, Ltd. Price 25*s.*)

INTENDED for first-year students at schools of building, the book deals with brickwork, floors, partitions, plumbing, stairs, windows, and so on in an elementary manner. The student would no doubt be better pleased with a less extravagant production at a lower price; this could be easily done by smaller drawings, smaller type and margins, and the avoidance of unnecessary words, for example, the definition "Square-edge boarding—Boarding with squared edge, i.e. without tongue and groove."

**"Specification for Road and Bridge Works."** (London: H.M.S.O. Price 5*s.*)

THE revised edition of this Specification, prepared by the Ministry of Transport and Civil Aviation, includes new clauses concerning lean concrete granular bases, premixed waterbound macadam bases, the use of soil-cement, and dense tar surfacing. The clauses relating to concrete carriageways have been revised; those relating to bridge works are unchanged.

THE UNIVERSITY OF LEEDS

BURSARIES

IN CONCRETE TECHNOLOGY

Applications are invited for Bursaries in Concrete Technology tenable from 1st October, 1958. The value of the Bursaries is £400 per annum, out of which the University fees have to be paid. They will be awarded for one year and may in certain circumstances be renewed for a second year. Applicants must hold a degree in Engineering, or its equivalent. The course will include post-graduate lectures, design, drawing and laboratory work. Applications, giving full details of qualifications and experience, must be received by the Registrar, The University, Leeds, 2, not later than 1st May, 1958.

IMPERIAL COLLEGE OF

SCIENCE AND TECHNOLOGY

Department of Civil Engineering

**Bursaries in Concrete Technology**

NOTICE IS HEREBY GIVEN that the election to Bursaries in Concrete Technology tenable as from October, 1958, will take place in June, 1958.

Candidates must hold a degree in Engineering at the time of taking up the award, and must also have a good knowledge of the theory of structures.

Bursaries are of the value of £460, out of which the College Tuition Fee of £64 has to be paid; the amount may be increased to a maximum of £760, depending on the qualifications and length and nature of industrial experience. The Bursary is for the normal College session extending from October until the following June.

The course will be postgraduate and Bursars who successfully complete the course will be eligible for the award of the Diploma of the Imperial College (D.I.C.).

Applications must be received on or before 1st June, 1958, by the REGISTRAR, IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY, Prince Consort Road, London, S.W.7, who will, on written request, send full information and application forms.

# A Proposed Mono-rail in London.

## REINFORCED AND PRESTRESSED CONCRETE CONSTRUCTION.

A PROPOSAL has been made by Air-Rail, Ltd., for the construction of a mono-rail between London Airport and central London with high-speed coaches travelling on elevated prestressed concrete beams, the sides of which would be gripped by horizontal wheels under the coaches. The vehicles would also be equipped with road wheels, so that they could be driven like buses at the airport. The journey would take about fifteen minutes.

The coaches would be driven by Diesel engines and be able to operate singly or in trains of two or three coaches. They would be 40 ft. long, constructed of magnesium alloy and plastic, and able to carry 50 passengers and their luggage; the weight when fully laden would be 12 tons. The suspension would be by metal-bonded rubber bearings between the wheels and the bogies, and by auxiliary air-bag suspension between the bogies and the coach. The capacity of the mono-rail would exceed 4000 passengers per hour in one direction, and the cost is estimated to be not more than £8,000,000.

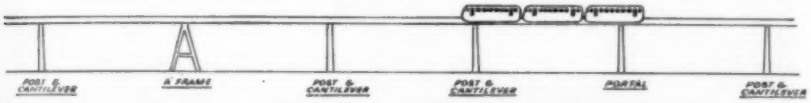



**Fig. 1.—Part Elevation of Proposed Mono-rail Structure.**

**Fig. 2.—Model of a Passenger Coach.**

The two-way track would consist of two prestressed precast hollow beams supported on reinforced pillars, rectangular frames, and A-frames (*Fig.* 1). The beams would be made in sections 10 ft. long, and assembled on the site to form beams of 100 ft. span. The rolling-stock would consist of passenger coaches (*Fig.* 2) and goods vehicles capable of travelling at 100 miles per hour; this limit is controlled by the performance of the tyres, but the track would be designed for speeds up to 250 miles per hour.

Messrs. R. Travers Morgan & Partners are acting as consulting engineers to Air-Rail, Ltd., whose directors are Sir Alfred Bossom, M.P., Mr. J. G. Lowe, Mr. E. C. Beck (Messrs. John Mowlem, Ltd.), Major-General R. F. K. Belchem, D.S.O., C.B.E. (Tube Investments, Ltd.), Mr. E. G. B. Bush (Concrete, Ltd.), Mr. V. C. Ellison (Associated Portland Cement Manufacturers, Ltd.), Lt.-Col. G. W. Kirkland,

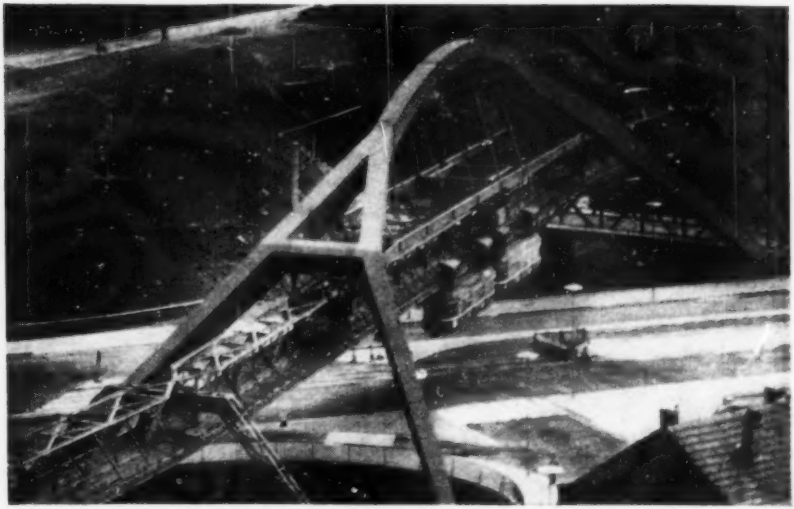


**Fig. 3.—A Mono-rail with Suspended Coaches in Germany.**

M.B.E. (Messrs. R. Travers Morgan & Partners), General Sir William Morgan, G.C.B., D.S.O., M.C. (Gloucester Railway Carriage and Wagon Co., Ltd.), and Lt.-Col. F. T. Orman, T.D. (Messrs. Orman & Partners). The address of the Company is 26 Ashley Gardens, London, S.W.1.

*Fig.* 3 shows a mono-rail of a different type recently constructed at Wuppertal, Germany.


## TABLES FOR THE DESIGN OF BEAMS AND SLABS ACCORDING TO B.S. CODE OF PRACTICE No. 114, 1957

In response to many requests, the article and tables by Mr. Jacques S. Cohen, in this journal for August, 1957, have been reprinted together with an additional table. These tables enable any rectangular section to be quickly designed by both the elastic method and the load-factor method, and the relative economy of the results of each method of design can be seen at a glance. The price of the reprint is 4*s*. (4*s*. 3*d*. by post); 1 dollar in Canada and the U.S.A.

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# New Type of Prestressed Bridge Deck.

A NEW type of deck has been used in rebuilding the bridge over the railway at Cooks Green in order to provide more headroom without lowering the track or raising the level of the road over the railway in connection with the electrification of the railway between Colchester and Walton.

The new deck is formed of slabs comprising three or four prestressed precast beams 18 in. wide by 10 in. deep placed side by side, encased with concrete to form slabs 33 ft. 5 in. long, 4 ft. 8 in. or 6 ft. 3 in. wide, and 16 in. deep (*Figs.* 1 and 2). Before encasing the precast beams solid rubber cores were placed longitudinally between the beams to form cable ducts when withdrawn (*Fig.* 1). The cables were tensioned by the Magnel-Blaton system, and the ducts grouted.

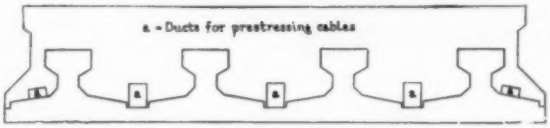


**Fig. 1.—Cross Section through Slab.**

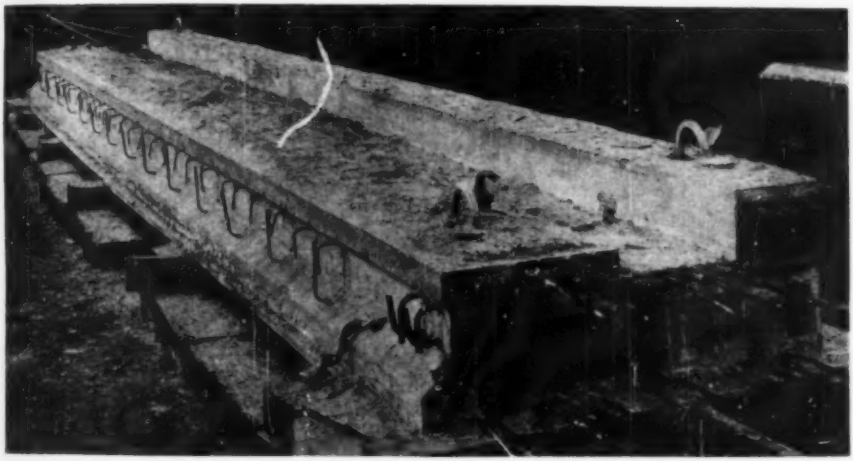

**Fig. 2.—Prestressed Slab Ready for Erection.**

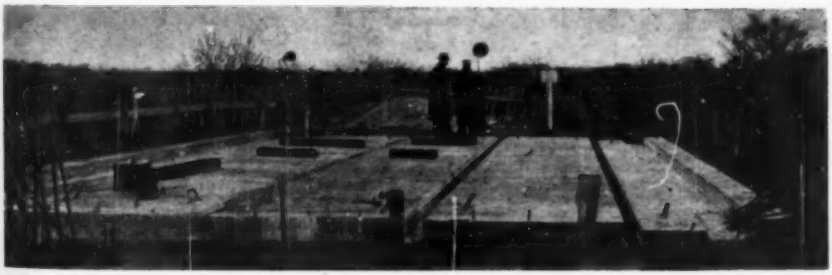

**Fig. 3.—Top of Deck.**

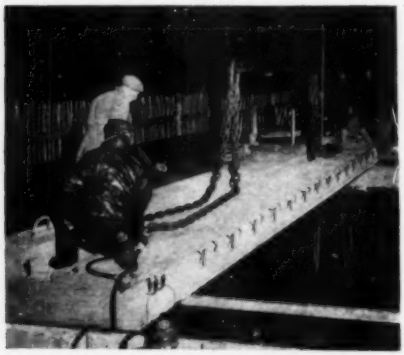

**Fig. 4.—Erecting a Slab.**

Transverse reinforcement projected into the channels formed at the sides of the slabs, which were filled with concrete cast in place. The resulting slabs, although only 16 in. deep, are capable of carrying the prescribed Ministry of Transport load with an adequate margin. The slabs were erected by crane (*Fig.* 4). Views of the deck, which is on a skew, are shown in *Figs.* 3 and 5.

**Fig. 5.—Underside of Deck.**

The bridge was designed under the direction of Mr. A. K. Terris, Chief Civil Engineer of the Eastern Region of British Railways, and the contractors were Messrs. Wellerman Bros., Ltd.

## *FIFTY YEARS AGO.*

From "Concrete and Constructional Engineering", March–April, 1908.*

Uses of Reinforced Concrete.—In the appendix of a book issued during 1907, 450 structures are enumerated, exclusive of floors (which are too numerous to mention), that have been recently built of this material in the United Kingdom, and from the notices that appear in the Press it will be seen that it has been used for almost every conceivable purpose, for bridges, baths, and barracks, churches, chimneys and conduits, for foundries and factories, for piers and palaces, power stations and post offices, for sewers and silos, for seawalls and swimming baths, for reservoirs and railway-sleepers, for walls and wharves, weirs and warehouses, for aqueducts and viaducts, etc. For the huge masts that are required to equip the great wireless telegraphy station, designed to communicate with America, reinforced concrete has been suggested, and among the more recent uses to which it has been applied to meet an up-to-date requirement reference might be made to the Brooklands Motor Track, where it spans the river Wey.

Reinforced Concrete for Warships.—An Italian correspondent writes that at a meeting of engineers held in Genoa on January 18 an Italian naval engineer lectured on a proposal to protect large ships with reinforced concrete instead of steel armour plates. He said that this system had been used with excellent results for land fortifications. While following the operations of the Russo-Japanese war in the Far East and near Port Arthur, he had been impressed by the resistance offered by Japanese casemates, which were thus protected, against heavy projectiles. The weight of such plates would be about the same, and the space required was not more than with metal armour, while the cost would be substantially less. The Ministry of Marine has directed that some of this concrete protection shall be exhaustively tried at the Muggiano. [Precast concrete slabs were used to protect the bridge and other parts of merchant ships during the war of 1939–1945.—Ed.]

* "Concrete and Constructional Engineering" appeared in alternate months until September, 1909.